may be assumed at 1,200,000 lbs. per sq. in., which is the value commonly used for pine.

Deflection.

In addition to strength in bending and shear, form members must be designed so that a safe maximum deflection will not be exceeded.

This may be taken at $\frac{1}{8}$ in. for sheathing, using the full live load of 75 lbs. per sq. ft. For joists and beams carrying joists the deflection should not exceed $\frac{1}{8}$ in. for dead load and a live load of 40 lbs. per sq. ft. For members under horizontal pressure, deflection should not exceed $\frac{1}{8}$ in.

Deflection considerations will generally govern the thickness of sheathing.

Actual and Nominal Timber Sizes.

It must be carefully noted that the actual dimensions of dressed wood will be less than the nominal by an amount varying from $\frac{1}{8}$ in. to $\frac{1}{2}$ in. This must be allowed for in design, as it makes a considerable difference to the strength of a timber, especially in the small sizes.

If the dressed sizes are not specified when ordering the following allowances should be deducted:

For sheathing up to 2 in. in thickness, deduct $\frac{3}{16}$ in. from the nominal thickness;

For timbers 2 in. by 4 in. up to 6 in. by 6 in., deduct $\frac{1}{4}$ in. from each dimension;

For timbers larger than 6 in. by 6 in., deduct $\frac{1}{2}$ in. from each dimension.

This rule will be followed in all tables and calculations, as it will give safe values for either dressed or undressed timber.

Accuracy.

Too great a refinement in design is not necessary, and is a waste of time. Exact calculations of bending moment are useless when so many assumptions are made as to live loads, stresses, quality of material, and especially workmanship on the job, and approximations are sufficiently close.

Sizes should be chosen that are sufficiently strong, remembering that actual construction in the field will not be as accurate as office calculations.

To design formwork intelligently a knowledge of the loads and pressures caused by wet concrete and the safe allowable stresses on timber is necessary, together with some knowledge of mechanics so that correct sizes can be chosen.

For those, however, who have had no training in mechanics, tables will be given covering ordinary conditions, from which the correct sizes to use can be obtained to suit the particular conditions.

DESIGN AND CONSTRUCTION OF FORMWORK FOR CONCRETE STRUCTURES

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INTRODUCTION

TO all those interested in reinforced concrete construction the economical design and construction of formwork is of great importance, as the cost of formwork is a large proportion of the total cost of the structure and the most difficult part to estimate. Also, the appearance of the finished structure and the speed with which it can be completed are mainly dependent on the efficient construction of the forms.

Considering its importance, the author has always considered it to be a subject more or less neglected. There is little information to be found on formwork in any technical book in the English language, and what information exists is either out of date or applies to some special structure. There is no one book covering all phases of the subject and all types of structures. The author has therefore attempted to present the subject in a manner consistent with its importance, covering most of the problems which commonly confront the form-builder and all the structures which are now generally built in concrete.

The design of various members has been fully treated because, except in the case of a few large specialist firms, this side of the subject is usually neglected; it is generally left to the foreman carpenter to choose the sizes and spacing of members. For safety and economy this should be a matter of design, with a knowledge of the strength of materials and the action of loads, and not be left to guesswork or rule-of-thumb methods.

The design data and tables are conservative, as it is better to be sure that the forms will be safe and will give good lines on the finished structure rather than to save a little material.

The author has not attempted to cover all the numerous methods of framing and constructing details, but has selected those methods which he considers from his own experience will give the best results. In this he has had the help of superintendents of construction who have spent many years studying the various methods of building forms.

Planning the work and standardisation have been emphasised in the interests of economy; haphazard methods are very common and expensive.

The author hopes the book will interest those contractors who have hitherto avoided reinforced concrete work because of their lack of knowledge of estimating and building formwork. He has also had in mind those who, while not having to build the forms themselves, are yet responsible for the finished structure, such as architects, resident engineers, and clerks of works, all of whom at some time or other have to pass

judgment on the strength of forms, on the time for stripping, and on the appearance of the concrete. Too many reinforced concrete structures are designed without any regard for the cost of formwork, so it is to be hoped that engineering students will study the subject together with that of reinforced concrete in order to be able to produce the most economical structure possible.

Considerable space has been devoted to steel forms because with the increasing cost and scarcity of timber their use is rapidly increasing. With the extending use of rapid-hardening Portland cements it is quite possible that steel forms will entirely replace wood forms, as owing to the greater speed of stripping less material will be required, the forms being used many times over in the same structure. As a rule, forms may safely be stripped from rapid-hardening Portland cement concrete structures in three or four days, while with aluminous cement this time can be reduced to twenty-four or thirty-six hours.

A. E. WYNN.

CHAPTER I.

FORM BUILDING IN GENERAL.

EVERY kind of reinforced concrete construction requires first of all the building of moulds in which to pour the concrete. These moulds, or, as they are termed, " formwork " or " shuttering," are therefore a very essential part of the concrete contractor's work. If he is experienced in this class of work the formwork will present no especial difficulties, as he will have developed a system of his own, worked out from his years of experience. But to the contractor who is just going in for this kind of work, or who only gets a concrete job occasionally, and to the owner who does his own construction, the method of designing and building the forms may not be familiar.

If there were a more universal knowledge of formwork erection there would be a greater use of reinforced concrete construction. Firms who have always used brick and steel in their construction work often have a prejudice against reinforced concrete because they feel they cannot get workmen familiar with form building, and that therefore the finished structure will either not look well or will cost too much money.

It is mainly for those who are unfamiliar with the subject that this book is written.

General Remarks.

There is very little literature on the subject, and probably no book in the English language devoted entirely to it. Being temporary and not permanent construction, it has always been left to the individual to develop his own methods.

However, reinforced concrete has been in use for so long now that the best of these methods have become standardised with firms who specialise in this construction. Although the general principles have been standardised, each foreman will probably have his own special preference for details. It is impossible to write about all the details of construction, because new ideas are being worked out every day, but there are certain details that are essential.

There is one best way to build any form, and that way is to use the timber available to the best advantage, having each member of the structure correctly proportioned to carry its part of the load with no waste of material, at the same time giving attention to a few construction details that will facilitate erection and stripping of forms.

Unskilled Work.

Lack of knowledge, and hence unskilled work, has often led to the collapse of a building. Lack of sufficient bracing and shoring, the use of timber not sufficiently strong to carry the loads, stripping too soon, etc., have caused many failures.

Even if there is no failure, the work will look bad and be a poor advertisement for this type of construction. Bulges in walls, sags in floors, wavy lines in beams and columns, and fins and ridges on the finished concrete are all due to unskilled knowledge of how to build the forms correctly, and can be avoided. Then there is the expense to the contractor who has to make good his work, no easy task after the concrete has once been poured.

Form Builders.

Specialist firms have some carpenters who do nothing but this class of work, and so have become experienced in the methods and details. This is as it should be. Form building is a trade of its own, and there are good opportunities for carpenters who are willing to train themselves in this class of work.

Since formwork is such an important part of concrete construction, by far the majority of concrete construction foremen are carpenters by trade.

The carpenter who is only used to indoor and finished work, however skilled he may be, is of little use in building forms, as it will take him a long time to realise that he is not building a permanent part of the work. He will waste so much time in accurate fitting and drive in nails that the only way to strip the forms will be with an axe.

Inexperienced contractors, whenever possible, should always employ two or three experienced form builders if they wish to produce rapid building.

Inspection.

When the forms are built ready for concreting they should always be carefully inspected by the architect or engineer, who should know the essential points to look for and should be able to tell at a glance if the work is satisfactory and able to carry the loads.

The carpenter foreman, in the rush of the work, will often overlook some important detail, leave out a brace or shore, forget to straighten up his lines, omit some tie wires, etc., with resulting sags and bulges and poor lines, if not failure.

During concreting, too, it is specially important to have at least one carpenter watching the forms, tightening wedges, adding braces, looking for weak spots, on the alert all the time for emergencies. "Prevention is better than cure" is especially true of concrete work, and many weak places have been rectified in time by this means. It will be too late after the concrete is poured.

Economy.

The foregoing remarks will all lead to economy, but there is also economy in design.

Design should not be left to the individual foreman or carpenter, unless he is very experienced, as he seldom has a knowledge of the mechanics of materials. He may, through years of experience, choose the right size and spacing of timbers to carry the loads, but it is more by good luck than good management, and he has probably had to learn through his mistakes.

It is almost always the case when something out of the ordinary is to be constructed, with extra heavy loads, that if the design is left to the field organisation it will be too weak. The size and spacing of joists, girts, studs, shores, yokes, wales, etc., should always be worked out in the engineer's or contractor's office. This will lead to the greatest economy in material.

The details of framing can be left to the practical mechanic in the field, who, however, should be taught and trained in the observance of the essential points.

Economy in stripping is very important. However well a form may be built, its ultimate success depends upon the speed and ease with which it can be stripped. This is a point which is often overlooked, and it is here that most of the essential details of framing occur.

The economy of being able to use the timber over and over again is obvious. The success of this, however, depends on the original design and method of framing. When making the design it is necessary to look ahead and observe how many times the timber can be used over again, and what changes will be necessary. This means unit or panel construction.

If the formwork for the first floor of a building is so built that it has to be all torn apart to build the second floor there is no economy. The first floor should have been built, as far as possible, in units that could be used on the second floor with few or no changes.

Stripping.

Experience will tell how long to leave the forms before stripping, but in general this should be decided by the architect or engineer. It will depend mainly upon weather conditions, and will vary with different parts of the structure. The knowledge of how to re-shore after stripping is very important when speed is necessary, and it is usually important for economy. The experienced contractor who can re-shore without bringing any strain on the green concrete can be allowed to strip much earlier than a contractor who has not had that experience.

If re-shoring is to be necessary it must be allowed for in the design, which must be made so that certain shores can be left in place while the rest of the forms are stripped. Too many failures have resulted from wrong stripping to allow it to be left to chance.

Systematisation.

On big work, and small work if the workers are inexperienced; the design should be made in the office by an experienced man, and all the main forms detailed on paper and given out to the foreman. This will ensure economy and adequate strength, and will relieve the foreman of much responsibility and detail work and enable him better to look after the actual execution of the work.

It will also lead to systematisation, using the same methods on different jobs, after, of course, the method has been proved to be the best. It will save endless argument with foremen, who generally wish to build the forms in their own way.

Better costs can be kept and better comparisons made between foremen and different gangs of carpenters when they are all following the same methods.

It is a great advantage for a foreman to be able to give the carpenters a sketch and tell them to "make up so many of these panels, column or beam sides, etc." With no system the foreman will be running all over the job, and the carpenters will be standing still wanting to know what to do next.

Of course the man in the office, who makes the design, must also have a knowledge of how the forms are erected.

Framing Details.

Besides the main features of design there are many practical tips that are learnt by experience. Some of them have become standard practice and will be mentioned later.

An ingenious workman can save much money for his employer by discovering better methods of executing details.

Ordering Material.

Material should be ordered in the office and not on the job. Haphazard ordering of timber leads to waste. However much timber is sent to a job it will always be used up, as a carpenter will always choose new timber in preference to old. Even with careful economy on the job there is usually more timber used than estimated.

From experience or detail drawings the bill of material can be taken off and just the right amount ordered allowing for waste. The foreman should be told how much he is allowed for the job, and he will then exercise economy. The timber should not be sent to the job all at once, or there will undoubtedly be waste.

Cost.

The cost of formwork is the most difficult part of a reinforced concrete structure to estimate.

Unless a system is adopted the costs may vary enormously with different workmen. They vary greatly, too, with the carpenter-foreman,

for one man may be able to get twice as much work out of the men as another and he may also be able to plan his work better.

Systematic design and planning of the work on the job are the only things that will keep down costs.

As far as possible the number of man-hours required to perform various operations will be given. They cannot, of course, be exact and must only be used as a guide to estimating. Man-efficiency has enormously decreased since the war, and during the last few years it has been difficult to estimate how much a man would accomplish in a given time.

Old data on costs are quite unreliable now, and the only right method is for each contractor to keep his own costs and work out his own cost units. Cost keeping is a problem in itself, but it cannot be stressed too highly, especially on this class of work, if the contractor wishes to come out on the right side.

In estimating concrete work the job should be analysed into its component parts and each part priced separately at so much per square or lineal foot. Estimating formwork at so much per cubic yard of concrete or at so much per square foot of floor area, as is often done, although it forms a check on more detail calculations, may lead to serious losses.

General Requirements.

Strength is the first requisite; the forms must be strong enough safely to carry the dead load of the concrete plus a live load applied during concreting.

Durability and rigidity are of next importance; the forms must be stiff and able to withstand hard and repeated usage. Lines must be true, bulges and sags must be prevented.

Cheapness, consistent with strength, is what the contractor is interested in. This means economy of material and correct construction details.

Economy in treatment of the concrete surfaces depends upon the tightness with which the forms are built, preventing ridges and fins, which afterwards have to be chipped off, caused by seepage through cracks.

Engineer's Design.

Although the engineering design of the structure is not the concern of the form builder, for true economy it should be made in conjunction with him. If the builder and designer are the same firm there is no difficulty in this.

If, however, an independent engineer or architect makes the structural design he cannot consult beforehand with an unknown builder. He can, however, make his design so that the form construction will be of the simplest.

A very small change in design often results in large savings in the cost of forms. A little extra concrete is usually cheaper than changing

forms to save concrete. As few changes as possible in column and beam sizes from floor to floor mean a great saving in labour.

Where live loads decrease on the upper floors of a building it is generally better to keep the beam sizes below the slab the same as on the heavier floors.

Suiting the design to the commercial sizes of dressed timber is rarely done, though it can be just as easily as not. It is common practice, for instance, to give beam widths in even inches, while dressed timber will make up into widths of odd inches with perhaps a fraction over.

This does not matter if the engineer will accept a beam width that is perhaps an inch less than specified, but he rarely will, and should not do so unless allowed for in his design. For a beam specified as 12 in. wide it would require a beam bottom made of a 2 in. by 12 in. dressed to, say, 11½ in. with the addition of a ½-in. strip, or it would require two 2 by 6's dressed to, say, 5½ in., giving a total width of 11 in., with the addition of a 1-in. strip. The design could have been made so that a width of 11 in. would be sufficient, and so save piecing out.

Similarly, a beam specified as 8 in. wide should be 7½ in. so that 2 in. by 8 in. dressed timber could be used.

Much time and labour are spent in filling out to meet even dimensions because the designer had not thought of economy in forms. Many designs are made which may be perfectly good from an engineering standpoint but which are costly in form building.

CHAPTER II.

MATERIALS, LOADS, PRESSURES AND STRESSES.

Timber.

THE most easily obtainable timber for formwork is pine, either Norway or American Southern pine, which is available in all sizes and is easily worked and cheap in comparison with other woods. Where strength is required, particularly in the larger size timbers, there is no wood equal to American pitch pine or Southern long leaf pine.

When it can be obtained spruce is one of the best timbers for all-round use. Hemlock is not desirable, as it is coarse grained and liable to curl.

Fir is now being used to some extent in America, and Douglas fir timbers are almost equal to long leaf pine.

Freedom from knots and coarse grain is desirable, as these will show on the finished concrete, for which reason soft white pine is one of the best timbers to use for mouldings, cornices, bridge parapet walls, etc., where an extra smooth finish is required. Soft white pine is, however, too expensive and has too little strength for form timber generally.

Hardwoods are not used for formwork except as caps and wedges under or over posts, where they are used to increase the allowable compression across the grain and so often allow a smaller post to be used. Hardwoods are difficult to work and nail.

Partially seasoned timber is the best for formwork, as if it is too dry it will tend to swell from absorption of moisture, while green lumber will tend to dry out and shrink in hot weather, causing fins and ridges on the concrete.

Timber may be rough or dressed, though workmen do not like to use rough wood. It may be dressed in various ways, such as all four sides, one side and one edge, one side and two edges, etc. Usually it is best to use timber dressed on all four sides, as it will then be of more uniform size and is more easily adaptable for different purposes.

Wood of any one size should be dressed to a uniform thickness, so that the pieces will match up ; this is particularly important with sheathing, as otherwise labour will have to be spent in planing down the joints. Joists and studs, too, if they are not of uniform thickness, will cause considerable trouble in fitting.

Sheathing, 1 in. to 2 in. thick, may be tongued-and-grooved, square, or bevelled edge. Tongued-and-grooved gives the best results, while a bevelled edge is good if the wood is very dry, as when built up it will

not buckle so easily when swelling. Square-edged timber is usually only used in the heavier thicknesses.

Thicknesses of timber will depend on the available supply and the loads to be carried, but more often on the former, as any ordinary size can be used to advantage by adjusting the spacing of the supports.

In general, for floor sheathing, 1 in. dressed down to $\frac{13}{16}$ in. is the usual practice; for wall sheathing and beam and column sides the thickness may vary from 1 in. to 2 in. stock dressed $\frac{3}{16}$ in. or $\frac{1}{4}$ in. Beam bottoms are generally 2 in. stock.

Joists may be any size from 2 in. by 4 in. to 3 in. by 10 in., 2 in. by 6 in. being the commonest size; column yokes are usually 3 in. or 4 in. by 4 in.

Studs and wales vary from 2 in. by 4 in. to 6 in. by 6 in.; posts from 3 in. by 4 in. to 6 in. by 6 in.

In ordinary work the smaller sizes mentioned above are used, extra strength being obtained by doubling up.

The lengths of timber ordered, when this can be specified, should be such that they can be used to the best advantage, with the least waste, a point which is often overlooked.

Sheathing can be ordered in random lengths, as it generally has to be cut up and short lengths can always be worked in.

Joists, studs, posts, beam bottoms, etc., where exact dimensions have to be met, should be ordered the nearest commercial length to the height or span required. If floor joists, for instance, are to span 5 ft. 6 in. they should be ordered in 12-ft. lengths for the least waste. Care in specifying the lengths is important, otherwise there will be a lot of short ends and a surprising percentage of waste. Timber cost is a big item in reinforced concrete construction, and it should be ordered and used with care.

Nails.

Common wire-cut steel nails are used, the most general sizes being *6d.*, *8d.*, *10d.* and *20d.*

Double-headed nails, if they can be obtained at a reasonable price, are an advantage as they can be drawn easily.

Wire and Bolts.

Tie wire for tying wall forms may be either number 8, 9 or 10 black annealed wire, number 9 giving the best service for ordinary work.

Steel or galvanised iron wire should not be used, as it is brittle, hard to handle, and too springy.

Bolts with washers and nuts are generally used in heavier wall construction in sizes from $\frac{1}{2}$ in. to $\frac{3}{4}$ in., usually with square heads and nuts. If they are to be drawn after use they should be well greased or fitted with sleeves.

Oil or Grease.

All forms coming in contact with the concrete, if the concrete is not to be plastered, should be well oiled or greased to allow easy stripping

and to prevent concrete adhering to and coming away with the forms.

If the concrete is to be plastered the plaster will adhere better if the surface is rough.

Special non-staining oil, made for the purpose, is the best to use, though soft soap and water is satisfactory.

Patented Articles.

There are many patent devices for facilitating the erection and stripping of forms, such as clamps, column yokes, adjustable shores, etc. Most of these are satisfactory and will save time and labour on a large job, but taking into account their first cost and the fact that they are easily lost, the ordinary methods are sometimes the cheapest in the end. Some of these special devices will be mentioned later.

Loads.

The load to be carried by formwork is the weight of the wet concrete and the forms themselves and a live load which allows for impact, wheeling over the forms, etc., and is therefore a construction load.

The weight of the forms can be neglected, as it is small compared with the other loads.

To simplify calculations, the weight of concrete may be taken as 144 lbs. per cubic foot. It is then only necessary to multiply the thickness of a floor by 12 to get the weight per square foot, or to multiply the depth and width of a beam together to get the weight per lineal foot.

For instance, a 5-in. slab will weigh 60 lbs. per sq. ft., and a beam 10 in. wide by 18 in. deep will weigh 180 lbs. per lineal foot. Inclined slabs, such as often occur in power-house floors and saw-tooth roofs, will cause an overturning movement to the top of the posts, and this must be taken care of by adequate bracing.

The assumed construction live load is generally taken as 75 lbs. per sq. ft. of floor. This value should always be used in designing floor sheathing and joists ; but when calculating the deflection of joists it can be reduced to 40 lbs. per sq. ft., as it will only exist during concreting and then only for short periods. After a bay is concreted there will only be the dead load to be carried, and 40 lbs. per sq. ft. will allow for any accidental loading.

Live load on ledgers is often omitted when calculating deflection.

For the good of both the concrete and the forms, the piling of timber, steel, cement, etc., on freshly-poured concrete should not be allowed. If it is known beforehand that, owing to confined space, some material must be placed on the concrete the day after it is poured, then the forms should be made extra stiff and should be designed to carry this loading with a small deflection.

Pressures.

In vertical sections, such as columns and walls, a horizontal pressure will act on the forms due to the hydrostatic head of the wet concrete.

This is the pressure which causes most of the bulges and collapse of forms.

It is a much debated point what pressures should be allowed for in the form design, and it is a subject about which there is as yet not much definite information. The pressure will depend on the rate of filling and the temperature.

The faster the forms are filled, and also the lower the temperature, the greater will be the pressure, because the concrete does not set so quickly and thus relieve the pressure. If a wall were poured so slowly that each layer set before the next layer were poured, when the wall was full the pressure at the bottom would be no greater than at the top. This is the principle of moving forms with which elevators and bins of all kinds and sometimes walls are built. The form is raised at about the same rate as the concrete sets, so that each layer supports the layer above.

Concrete at a low temperature will set slowly, and for a given rate of pouring the pressure may be 50 to 75 per cent. greater than when the temperature is twice as high. For this reason forms should be built stronger in very cold weather than in the summer, and not stripped so soon.

Temperature considerations are, however, not often taken into account in designing forms, except in very cold weather, and the pressure is assumed at an average temperature allowing an ample factor of safety.

Rate of pouring is more important, and must be taken into account. Many wall and column failures have occurred by pouring concrete too fast, or, if it was necessary to pour fast, by designing the forms too weak. If a 1-cu. yd. mixer is used, naturally the wall will be filled faster than if a ⅓-cu. yd. mixer is used, and hence the forms must be made correspondingly stronger.

Vertical sections should always be poured as slowly as is consistent with economy, and in long layers about 12 in. thick.

Concrete in heavy walls and piers, in which large stones or " plums " can be embedded, will always exert less pressure on the forms than when the stones are omitted, because true hydrostatic pressure will not exist.

Since the outward pressure depends mainly on the rate of pouring, column sides will be under greater pressure than wall sides, since they are filled faster. Column forms, however, should never be filled to the top without a break; instead, each batch of concrete should be distributed amongst several columns. For column forms a hydrostatic pressure equivalent to that due to a liquid weighing 125 lbs. per cu. ft. should be used; that is, the pressure on any yoke per lineal foot will be the depth from the top multiplied by 125 multiplied by the spacing apart of the yokes.

Small low walls may be poured as fast as columns and the same pressure should be used, but the higher and wider the wall the slower

will it be filled and the pressure exerted at any point will correspondingly decrease.

With ordinary thicknesses of walls it is simpler, for calculation purposes, to consider the height of the wall instead of the rate of pouring, the one being proportional to the other. The width does not affect the pressure, but does influence the speed of pouring, so that thin walls should be poured more slowly than thick walls, assuming that they are not designed for fast pouring. To some extent in narrow walls friction and arch action reduce the hydrostatic pressure.

The following is a useful guide to follow in designing ordinary walls in order to be on the safe side:—

Height of wall.	Pressure will be equivalent to that of a liquid weighing
Less than 5 ft.	145 lbs per cu. ft.
5 ft to 10 ft	125 ,, ,, ,,
10 ft. to 20 ft.	100 ,, ,, ,,
Over 20 ft.	75 ,, ,, ,,

The consistency of the concrete also affects the pressure, increasing with the increase of the percentage of water in the mix.

Stresses.

As formwork is only temporary, higher unit stresses may be allowed than would be permissible in permanent work.

For yellow pine, spruce, fir, and timbers of equal strength a maximum fibre stress of 1200 to 1400 lbs. per sq. in. for bending may be used, the former value being the most common and is conservative. For horizontal shear 200 lbs. per sq. in. should be used, and for bearing or crushing across the grain 400 lbs. per sq. in.

For American long leaf or pitch pine these stresses may safely be increased by 50 per cent.

For posts with square end bearing the maximum allowable compressive stress should be 1000 lbs. per sq. in., to be reduced according to the ratio of the height or unsupported length to the least diameter by the formula

$$\text{safe unit stress} = 1000\ (1 - h/80d),$$

where h is the unsupported length and d the least dimension of the cross-section.

For ordinary conditions this gives a unit stress of about 750 to 800 lbs. per sq. in., which is twice that allowable for compression across the grain. Therefore the size of posts will be generally limited by compression across the grain in the timbers they carry.

If oak or other hardwood caps or wedges are inserted between the load-carrying member and the top of the cap, the allowable unit compressive stress across the grain may be increased 50 per cent.

Modulus of Elasticity.

For the purpose of calculating deflections the modulus of elasticity

may be assumed at 1,200,000 lbs. per sq. in., which is the value commonly used for pine.

Deflection.

In addition to strength in bending and shear, form members must be designed so that a safe maximum deflection will not be exceeded.

This may be taken at $\frac{1}{8}$ in. for sheathing, using the full live load of 75 lbs. per sq. ft. For joists and beams carrying joists the deflection should not exceed $\frac{1}{8}$ in. for dead load and a live load of 40 lbs. per sq. ft. For members under horizontal pressure, deflection should not exceed $\frac{1}{8}$ in.

Deflection considerations will generally govern the thickness of sheathing.

Actual and Nominal Timber Sizes.

It must be carefully noted that the actual dimensions of dressed wood will be less than the nominal by an amount varying from $\frac{1}{8}$ in. to $\frac{1}{2}$ in. This must be allowed for in design, as it makes a considerable difference to the strength of a timber, especially in the small sizes.

If the dressed sizes are not specified when ordering the following allowances should be deducted:

For sheathing up to 2 in. in thickness, deduct $\frac{3}{16}$ in. from the nominal thickness;

For timbers 2 in. by 4 in. up to 6 in. by 6 in., deduct $\frac{1}{4}$ in. from each dimension;

For timbers larger than 6 in. by 6 in., deduct $\frac{1}{2}$ in. from each dimension.

This rule will be followed in all tables and calculations, as it will give safe values for either dressed or undressed timber.

Accuracy.

Too great a refinement in design is not necessary, and is a waste of time. Exact calculations of bending moment are useless when so many assumptions are made as to live loads, stresses, quality of material, and especially workmanship on the job, and approximations are sufficiently close.

Sizes should be chosen that are sufficiently strong, remembering that actual construction in the field will not be as accurate as office calculations.

To design formwork intelligently a knowledge of the loads and pressures caused by wet concrete and the safe allowable stresses on timber is necessary, together with some knowledge of mechanics so that correct sizes can be chosen.

For those, however, who have had no training in mechanics, tables will be given covering ordinary conditions, from which the correct sizes to use can be obtained to suit the particular conditions.

CHAPTER III.

THEORETICAL DESIGN OF FORMS.

Designing for Vertical Loads.

THE vertical loads to be designed for are the weight of the concrete and the live load on the floor. The different members entering into the forms will be sheathing or lagging, joists, ledgers or girts carrying the joists, and posts. The formulæ for calculating the size and spacing of these members will be given so that the general principles of design may be understood and special cases worked out quickly.

For ordinary conditions, however, the sizes and spacing can be taken directly from tables based on these formulæ.

For members subject to bending, the four considerations are: (1) strength to resist bending moment; (2) strength to resist horizontal shear; (3) maximum allowable deflection; and (4) minimum allowable bearing on support.

The design will also depend on whether the member is a single span or extends over more than two supports. A member that has one or more supports in its length in addition to the end supports is stronger for the same load than if it were simply supported at the two ends only, because the bending moment and deflection will be less.

The design of the posts will depend on their height and size and on the crushing of the grain of the timber supported.

Symbols.

In the formulæ here given the following symbols will be used:—

w = uniform load per lineal ft. in lbs. (dead + live).
w' = uniform load per sq. ft. in lbs. (dead + live).
P = concentrated load in lbs.
p = pressure in lbs. per sq. ft.
p' = pressure in lbs. per lineal ft.
l = span in ft.
b = breadth of member in inches.
d = depth of member in inches.
s = spacing of member in inches.
D = deflection of member in inches.
E = modulus of elasticity in lbs. per sq. in.
M = bending moment in inch pounds.
M_r = resisting moment in inch pounds.
f = maximum fibre stress in lbs. per sq. in.

v = horizontal shearing stress in lbs. per sq. in.
V = total shear at end of a member.
I = moment of inertia in ins.^{4} = $bd^3/12$.
h = height in ft.

Bending Moment.

It is usual practice to design members as partially continuous when they extend over more than two supports, taking the bending moment as $M = wl^2/10$ for uniform load; and if the member is simply supported on two supports only as $M = wl^2/8$. Floor, wall, and column sheathing will always be partially continuous.

Ledgers carrying joists and wales supporting studs are also usually designed as partially continuous, as generally long timbers are used with intermediate supports.

Joists and column yokes are simply supported as a general rule. It will be seen that one bending moment is eight-tenths of the other. If the loads are concentrated, as with ledgers and wales, it is still close enough to assume that the bending moment for a continuous member is eight-tenths that for a member supported at two points only.

Bending.

The first consideration is strength in bending, although this will not always govern the size of a member.

The bending moment on a member must equal its resisting moment or $M = M_r$, and, from the fundamental formula for bending, $M_r = \dfrac{I \cdot f}{d/2}$.

So for *uniform load and single span* $\dfrac{wl^2.12}{8} = 1200.\dfrac{bd^3}{12}.\dfrac{2}{d}$.

For given size of member, maximum span $= l = \sqrt{\dfrac{133 \cdot 33\, bd^2}{w}}$. (1)

For given span and width of member, $d = \sqrt{\dfrac{wl^2}{133 \cdot 33 . b}}$. (2)

For *uniform load and continuous span*, $M = \dfrac{wl^2.12}{10} = 200\, bd^2$

and we have $l = \sqrt{\dfrac{166 \cdot 67\, bd^2}{w}}$. (3)

$d = \sqrt{\dfrac{wl^2}{166 \cdot 67 . b}}$. (4)

Spacing: $w = sw'/12$,

and for *single spans*, $s = \dfrac{1600\, bd^2}{w'.l^2}$ . . . . (5)

and for *continuous spans*, $s = \dfrac{2000.bd^2}{w'.l^2}$ . . . . (6)

Horizontal Shear.

The load carried by a member is transmitted to the supports, where it causes a shearing action on the member tending to tear the fibres apart. If the load is symmetrical the shear at each support will be half the total load on the member. On short spans and with heavy loads, where only a small depth is required for bending, it will often happen that the member will not be strong enough to resist shearing, and so the size has to be increased to allow a safe unit shearing stress.

The unit shearing stress $v = 1{\cdot}5.V/bd$ . . . . (7)

with $v = 200$, size of member required, $bd = \dfrac{1{\cdot}5\,V}{200}$ . . . . . (8)

Spacing for shear, for joists with uniform load, $w = sw'/12$, and $V = wl/2$, substituting in 8, $s = \dfrac{3200\,bd}{w'l}$ . . . . (9)

Deflection.

The deflection of form members must be limited or there would be wavy lines on the concrete, cracks in finished floor surfaces, etc. When the depth required for bending is small, the allowable deflection will usually govern the size to use. This is particularly the case with sheathing.

For *uniform load and single span*, $D = \dfrac{5}{384}.\dfrac{w}{12}\,\dfrac{(l{\cdot}12)^4}{E.I.}$ . . . . (10)

For *joists*, substituting $I = bd^3/12$, $E = 1{,}200{,}000$, $w = \dfrac{sw'}{12}$ and s from 5,

$$D \text{ (in ins.)} = \frac{0{\cdot}03\,l^2}{d} \quad . . . . (11)$$

If D is limited to $\frac{1}{8}$ in., max. span l (in ft.) $= 2{\cdot}04\sqrt{d}$ . . . . (12)

If D is limited to $l/360$ in., max. span l (in ft.) $= 1{\cdot}111d$ . . (13)

For continuous spans, the deflection can be assumed as the average of the deflection for a beam with fixed ends and a beam simply supported.

For *uniform load and continuous span*, $D = \dfrac{3}{384}.\dfrac{w}{12}\,\dfrac{(l12)^4}{EI}$ . . (14)

For sheathing, $D = \frac{1}{8}$ in., l (in ft.) $= 10{\cdot}25\sqrt[4]{\dfrac{d^3}{w}}$ . . . (15)

For *joists*, substituting as before, giving s value from 6,

$$D = 0{\cdot}0225\,\frac{l^2}{d} \quad . . . . (16)$$

If D is limited to $\frac{1}{8}$ in., max. span l (in ft.) $= 2{\cdot}357\sqrt{d}$ . . (17)

If D is limited to $l/360$ in., max. span l (in ft.) $= 1{\cdot}481.d$ . . (18)

SPACING FOR DEFLECTION:

$$D = \tfrac{1}{8} \text{ in., } \textit{single spans, } s = \frac{6667\, bd^3}{w'l^4} \qquad (19)$$

$$\textit{continuous spans, } s = \frac{11112\, bd^3}{w'l^4} \qquad (20)$$

$$D = l/360 \textit{ single spans, } s = \frac{1778\, bd^3}{w'l^3} \qquad (21)$$

$$\textit{continuous spans, } s = \frac{2963\, bd^3}{w'l^3} \qquad (22)$$

Bearing.

The ends of joists and ledgers must have sufficient area of bearing on their supports to prevent the crushing of the grain of the timbers. Joists will rest on ledgers nailed to the beam sides and centre span ledgers will rest on posts, and the size of these supports will depend on the allowable unit crushing stress across the grain, which = 400 lbs. per sq. in.

If V is the shear at the end of a member, or the load transmitted to the support, then

for *ledgers carrying joists*,

$$\text{thickness of ledger} = \frac{V}{400\, b} \quad . \; . \; . \; . \; . \; . \quad (23)$$

for *posts carrying ledgers*,

$$\text{area of bearing on post} = \frac{V}{400} \quad . \; . \; . \; . \; . \quad (24)$$

Concentrated Loads.

The above formulæ will not apply when the load is concentrated on a member at one or more points, as is the case with ledgers carrying joists.

The maximum bending moment will occur when there is a load at the centre of the span and the maximum shear when the load is at the edge of the span. For one load $M = Pl/4$.

The *bending moment* is calculated as follows for 5 concentrated loads s ft. apart (*Fig.* 1):—

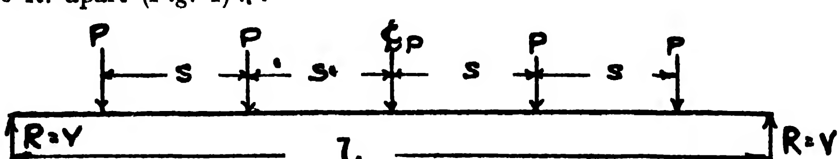


FIG. 1.

Load carried by supports $V = 5P/2$.

$$M \text{ at centre} = \frac{5P}{2} \cdot \frac{l}{2} - P.2s - Ps, \text{ or } M = P(5l/4 - 3s).$$

For three joists only, $M = P(3l/4 - s)$; and for seven joists, $M = P(7l/4 - 6s)$, and so on.

If the member is continuous the bending moment can be calculated in the above manner and then multiplied by $\frac{8}{10}$.

Having found the bending moment, this is equated to the moment of resistance of the member as before, or $M.12 = 200\ bd^2$ . . . . (25)

The *shear* is calculated, assuming one joist will be at the edge of the support, as follows (*Fig.* 2) :—

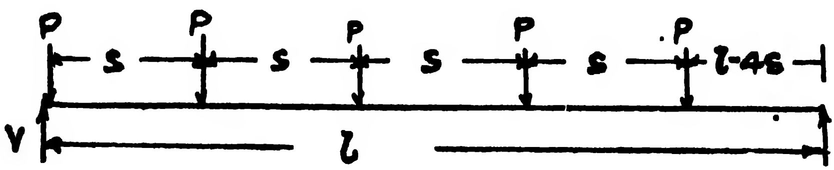


FIG. 2.

$$V = P + \frac{P(l-s)}{l} + \frac{P(l-2s)}{l} + \frac{P(l-3s)}{l} + \frac{P(l-4s)}{l}, \text{ etc.} \quad (26)$$

and unit shearing stress $v = 1{\cdot}5\ V/bd$ as before.

The *deflection* of beams carrying concentrated loads is more difficult to calculate than with uniform loading. An approximation is close enough for ordinary purposes.

For *single load and single span* (load at centre) $D = \dfrac{Pl^3}{48EI}$. (27)

For *single load and continuous span* (approx.) $D = \dfrac{5.Pl^3}{384EI}$ (28)

For TWO LOADS SYMMETRICAL ABOUT CENTRE LINE,

for *single span*, $D = \dfrac{Pa}{24EI}(3l^2 - 4a^2)$ (29)

for *continuous span* (approx.) $D = \dfrac{5Pa}{192E.I}(3l^2 - 4a^2)$. (30)

Therefore, by combining the above two cases, we have approximately, for any number of loads, adding the deflection produced by any pair of loads equidistant from the supports to the deflection due to a centre load (*Fig.* 3) :—

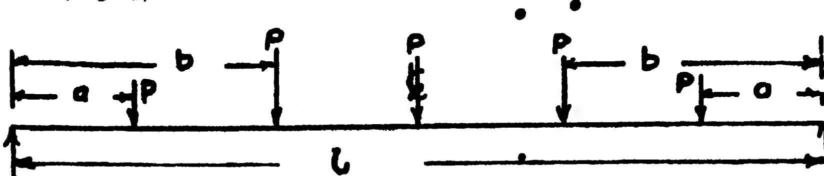


FIG. 3.

Single spans, $D = \dfrac{P}{48EI}(l^3 + 2a(3l^2 - 4a^2) + 2b(3l^2 - 4b^2) + \text{etc.})$ (31)

Continuous spans, $D = \frac{5}{384}\frac{P}{EI}(l^3 + 2a(3l^2 - 4a^2) + 2b(3l^2 - 4b^2)$ etc.) (32)

In above formulæ l, a, b, must be in inches.

Posts.

The load carried by the posts is easily calculated. It is assumed, with uniform load, that half the load carried by a member is transmitted to the post at each end, though this is not strictly true when the member is continuous over two spans or more, but the approximation is close enough. With concentrated loads one load should be placed over the post, and the load will be, for 1 span $= V$ from formula 26; for centre post of 2 spans load $= 2V - P$.

Special cases may arise where the member is cantilevered out beyond the post; in an extreme case the post might carry the whole load from the cantilever and also the adjoining span, but this is so rare in form building that it will not be considered. Deflection is so great with cantilevers that they should not be used.

Maximum safe load on post, $W = 1000(1 - h/80d).bd$ . . . (33)

The height "h" and least dimension "d" of post must first be assumed. The height may be the full height of the post, but more often half the full height, since posts above about 8 ft. high should be braced both ways at the centre.

Having found the safe load the post will carry, the crushing stress on the timber carried must be investigated, as this will usually govern.

The cross-sectional area of the post "bd" multiplied by 400 (the allowable unit stress) will give the total load that can be carried without crushing the fibres of the member supported.

This allowable unit stress is about half that allowed by formula (33) for ordinary conditions, and so not more than about half the load that a post would carry should be put upon it unless hardwood caps or wedges are inserted between the member and the top of the post and between the bottom of the post and the sill on which it rests, in which case the allowable unit stress across the grain may be increased to 600 lbs. per sq. in.

Designing for Horizontal Pressures.

The foregoing formulæ for the size and spacing of members are also applicable to the design of members under horizontal pressure, such as column and wall forms, but the loading will be different.

Column Forms—Sheathing.

As seen in Chapter II, the pressure at any point will depend upon the height of the column.

If w in lbs. is the weight of an equivalent fluid causing hydrostatic pressure (obtained from previous chapter), p the pressure in lbs. per

sq. ft., and h the distance of any point below the top of the column, then

$$p = wh \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (34)$$

It is assumed for simplicity that the pressure will be uniform between yokes and of an intensity equal to that at the lower yoke; this assumption is on the safe side.

As the sheathing will be continuous and the load assumed uniform, the bending moment can be taken as $M = ps^2/10$, where s is the spacing of the yokes (in ft.). Formulæ (3) and (4) will then apply with ps in place of wl.

The maximum span for a deflection of $\frac{1}{8}$ in. is given by formula (15) with "wh" in the place of "w." This will also be the maximum spacing of the yokes.

Yokes.

The total pressure per lineal foot on any yoke will be the pressure at the centre of the yoke multiplied by the spacing of the yokes at that point (approximately).

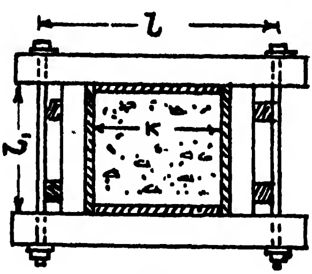


FIG. 4.

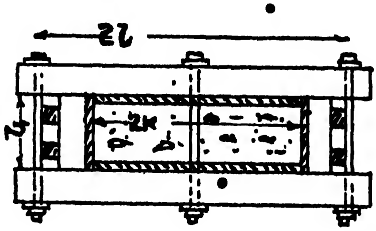


FIG. 5

If this total pressure is p', $p' = ps = whs$ . . . . (35)

This pressure will not extend over the whole yoke, since the span of the yoke will always be greater than the dimension of the column by an amount varying with the detail of construction, but will generally be about 12 in., that is (*Fig.* 4), $l - k = 12$ in. (approx.).

Unless the columns are very wide. say 5 ft., or over, requiring an intermediate bolt (*Fig.* 5), the yoke will be simply supported. Since the pressure does not extend over the whole span, and since the proportion of the span under pressure is variable with the size of yoke used, there is some difficulty in giving general formulæ and tables without making an assumption. However, $l - k$ varies within small limits, say from 8 in. to 14 in., and it is sufficiently close to assume a constant value of 12 in.; that is, the yoke can be assumed to be 12 in. longer than the dimension of the column. This enables us to write a simple general formula for the bending moment,

$$k = l - 1,\ p' = whs,\ M = \frac{whs}{8}(l^2 - 1) \quad . \quad . \quad . \quad (36)$$

If a centre bolt is used the bending moment on each half of the yoke would be a little less than given by the formula (36) (l in this case would be half the total span), but since columns of such width as to require a centre bolt do not often occur it is not worth while making a special case of this, as the spacing of the yokes would only change an inch or so.

A higher unit fibre stress can be allowed for column yokes, since there is no uncertain live load as on floor joists, nor is there the same vibration and impact, so we will assume the safe unit fibre stress as 1600 lbs. per sq. in. instead of 1200.

Bending.—Equating the bending moment to the moment of resistance,

we have $$\frac{whs}{8}(l^2 - 1).12 = \frac{1600.bd^2}{6}$$

$$w = 125\text{, and spacing } s \text{ (in ins.)} = \frac{17{\cdot}06\ bd^2}{h(l^2 - 1)} \quad . \quad . \quad . \quad (37)$$

This will give the spacing of the yokes in inches for any size yoke and depth below top of column and for any column width, remembering to take l one foot greater than the dimension of the column.

Horizontal Shear.—The end shear $V = whsk/2$, with a centre bolt

$$V = \frac{whs(l^2 - 1)}{4l}$$

unit shearing stress for single span, $$v = \frac{1{\cdot}5whsk}{2bd} \quad . \quad . \quad (38)$$

unit shearing stress for continuous span, $$v = \frac{1{\cdot}5whs(l^2 - 1)}{4l.bd} \quad . \quad (39)$$

Deflection.—The deflection can be found from formula (10) approximately, substituting whs for w and for s the value found from (37), and

we have, for single spans, $$D = \frac{{\cdot}04\ l^4}{d(l^2-1)} \text{ (approx.)} \quad . \quad . \quad . \quad . \quad . \quad (40)$$

If $D = \frac{1}{8}$ in., $$d = \frac{{\cdot}32\ l^4}{l^2 - 1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (41)$$

If $D = l/360$, $$d = \frac{1{\cdot}20\ l^3}{l^2 - 1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (42)$$

Usually a deflection of $\frac{1}{8}$ in. is allowed.

From these last two formulæ the maximum span for any depth of yoke may be found.

The above formulæ apply to the yokes that carry the connecting bolts. The other pair of yokes (*Figs.* 4 and 5) have a span l_1 equal to the side of the column, and this span is reduced by the wedges between the yokes and the bolts, so that the stress in them will always be less than in the long yokes and they can be made smaller if so desired.

Assuming full span l_1, the above formulæ will apply with $l_1{}^2$ put in the place of $l^2 - 1$.

Bolts.

The yokes will be held together by rods, bolts, or clamps. The tension in the bolt will equal the shear at the end of the yoke $= whsk/2$.

When bolts are threaded the effective area will be at the base of the threads; this area will be about two-thirds that of the gross area of the bar, and should be allowed for.

For threaded bolts, allowing a safe unit stress of 14,000 lbs. per sq. in. on the gross area, diameter

$$d = \sqrt{\frac{hsk}{175}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (43)$$

If the bolts are upset at the threaded end, and for plain round bars, the allowable stress may be 20,000 lbs. per sq. in., and then

$$d = \sqrt{\frac{hsk}{200}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (43a)$$

Upset bolts are seldom used.

The size of the bolt theoretically required is generally small and is not usually calculated, since less than ½-in. diameter bolts should not be used; common practice is to use ⅝-in. bolts for all columns, as they have to be sufficiently strong to withstand wedging against them without bending.

With very large columns, when a centre bolt is used, the load on this bolt should be calculated, as it may be necessary to use a ¾-in. bolt.

Washers.

Washers are necessary to distribute the tension in the bolt over sufficient area of the yoke so that the fibres will not be crushed in.

$$\text{So that area of washer} = \frac{whsk}{800} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (44)$$

This will give the net area required, and to it must be added the area of the hole for the bolt.

To find the thickness required, the washer is treated as a plate cantilevering about the edge of the head or nut and uniformly loaded.

If A is difference between area of washer and area of head or nut,
  d is side or diameter of head or nut,
  p is projection of edge of washer beyond head or nut,
  t is the thickness of washer,
  w is pressure on washer in lbs. per sq. in.

Then approximately for wrought-iron or steel washers,

$$t = \frac{1}{220}\sqrt{\frac{wAp}{d}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (45)$$

For cast-iron washers the thickness should be three times that given by formula (45). (Kidder's "Architects' Handbook.")

Theoretically the size of washer will change with the size and height

of the column, but this is not practical on the job so that a size is adopted as a standard which will suit all ordinary cases.

The following rule can be used as a guide :—

For columns up to 36 in. wide or square and 12 ft. high, use 3 in. sq. by $\frac{1}{8}$ in. washer.

For columns from 36 in. to 42 in. wide or square and 12 ft. to 16 ft. high, use $3\frac{1}{2}$ in. sq. by $\frac{1}{4}$ in. washer.

For columns from 42 in. to 48 in. wide or square and 16 ft. to 20 ft. high, use 4 in. sq. by $\frac{3}{8}$ in. washer.

With 4 in. by 6 in. or larger yokes do not use less than $3\frac{1}{2}$ in. by $\frac{1}{4}$ in. washers.

Wall Forms.

Sheathing.—The design of sheathing for wall forms is the same as for column forms, although in this case the sheathing will usually span horizontally instead of vertically.

The spacing of the vertical studs will depend on the strength of the sheathing, and since the pressure will be greatest at the bottom of the wall it is only necessary to find the maximum allowable span at this point. This is found in the same way as for floor sheathing, with wh in the place of w, so

for strength, $$\text{max. } l = \sqrt{\frac{166{\cdot}67\, bd^2}{wh}} \quad . \quad . \quad . \quad . \quad (46)$$

for deflection, $$D = \tfrac{1}{8} \text{ in., max. } l = 10{\cdot}25 \sqrt[4]{\frac{d^3}{wh}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (47)$$

$$D = l/360, \text{ max. } l = 14{\cdot}31 \sqrt[3]{\frac{d^3}{wh}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (48)$$

Studs.—Since the studs can be tied and braced at any point, the spacing will depend upon the strength of the sheathing, found from the above. The thickness of the stud can be added to the spacing found from the formula for sheathing, to give the actual spacing of the studs.

The spacing will be a minimum at the bottom of the wall and is kept the same for the full height, as studs are used in long lengths and it is not practical to vary the spacing with the pressure.

Accurately to find the bending moments on the studs would be a tiresome procedure, and the result obtained would not be worth the time expended, so assumptions can be made.

The studs are supported individually by internal ties of wires or bolts, or collectively by external wales, bolted or wired through the wall.

We will assume that the pressure is constant between any one wale or tie and the one next above it, with intensity equal to that at the lower one.

As for yokes, the allowable unit fibre stress can be increased to 1600 lbs. per sq. in.

At any depth in the wall the pressure on the stud will $= whl$, where l is the spacing of the studs (*Fig.* 6).

If s is the spacing of the ties, $M = \dfrac{whls^2.12}{10} \quad 1600.bd^2$

$$\text{or, for strength, max. span } s \text{ (in ins.)} = 179\sqrt{\frac{bd^2}{whl}} \quad . \quad . \quad . \quad . \quad . \quad (49)$$

$$\text{for deflection,} \quad D = \tfrac{1}{8} \text{ in., max. } s = 66\sqrt[4]{\frac{bd^3}{whl}} \quad . \quad . \quad . \quad . \quad . \quad (50)$$

$$D = s/360 \text{ max. } s = 75\cdot12\sqrt[3]{\frac{bd^3}{whl}} \quad . \quad . \quad . \quad . \quad (51)$$

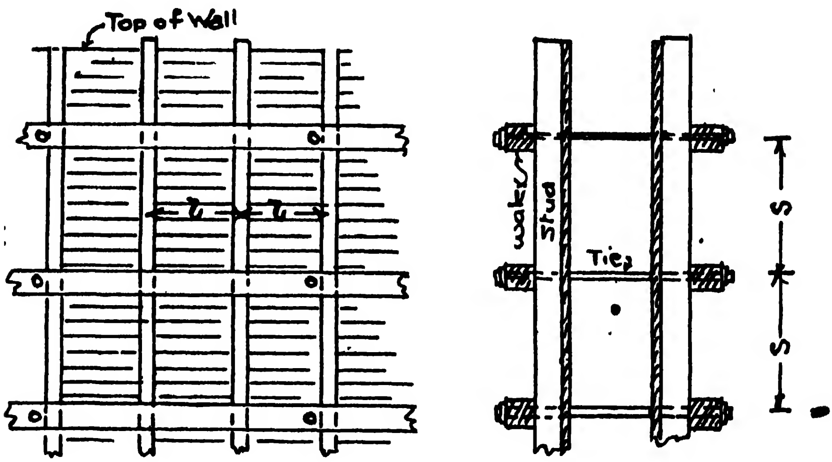


Fig. 6.

These formulæ will give the maximum spacing of the ties for any depth of wall and size of stud, or will give the spacing of the wales if used to support the studs.

Wales.—Wales are used to keep the forms in line, to support the studs, and to avoid the necessity of having to tie every stud. They are bolted through the wall or are braced from the outside, and transmit the load on the studs to the bolts or braces. They will be continuous members under concentrated loads, and are designed in the same way as ledgers carrying floor joists; the same formulæ and tables will apply, but instead of the loads carried by the joist it will now be the pressure transmitted by the studs or "*whls*," approximately, h being taken to the centre of the wale being designed (see formulæ 25 to 32).

This approximate method of finding the loads on the wales is close enough for ordinary purposes, the error being less than 10 per cent., but

occasions may arise when it is necessary to find a closer approximation, so the method will be given.

Calculation of loads on wales by more exact method.—The effect of continuity on the reactions at the wales will be neglected. The pressure of the concrete varies from zero at the top of the wall to a maximum at the bottom, being at any point proportional to the depth from the top, so that the pressure on a stud can be represented by a triangle (*Fig.* 7), *aej*, where *ac* represents the height of the wall and *aj* the horizontal pressure at the bottom.

If *b*, *c*, *d* represent the location of wales, the total pressure on the stud between any two wales, such as *bc*, will be represented by the area of the trapezoid *bcgh* and will act through the centre of gravity of this area at a distance x from *bh*.

The total pressure P on *bc* equals the average of pressures at b and c times *bc* or $P = (w.l.\,(cg + bh)/2)\,bc$, where l is the spacing of the studs.

If $ce = h'$ and $bc = h$ then $P = \dfrac{wlh(h + 2h')}{2}$ since $cg/bh = ec/eb$.

The centre of gravity of *bcgh* from *bh* is $x = \dfrac{h}{3}.\dfrac{h + 3h'}{h + 2h'}$.

The proportion of the load P transferred to c is therefore

$$P.x/h = wlh.\frac{h + 3h'}{6} \quad . \quad . \quad . \quad . \quad . \quad . \qquad (52)$$

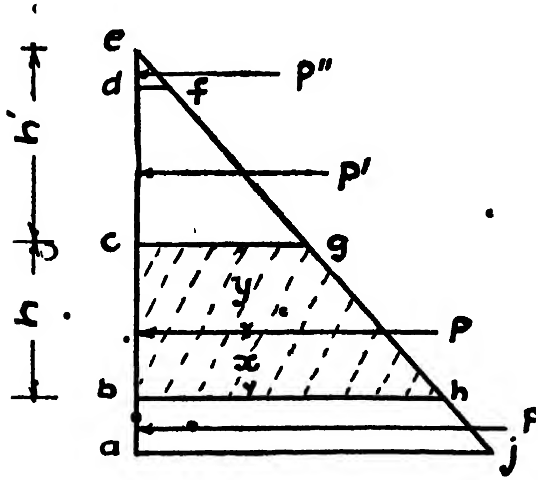


FIG. 7.

and the proportion of the load P transferred to b is

$$P.y/h = wlh.\frac{2h + 3h'}{6} \quad . \quad . \qquad (53)$$

The reaction at c from P' is similarly found, and the two reactions added together give the load on the wale at c.

It is assumed that the whole load on the stud below the lowest wale is carried by that wale, and that the whole load on the stud above the highest wale is carried by that wale.

The pressure P'' on de is $wl.de^2/2$, the whole load being carried at d.

Braces.—Theoretically a wall form could be built without any external braces, because the ties take all the pressure, but the impact of the concrete would soon throw the wall out of line, so that some braces should always be used.

The distance apart of the braces is largely a matter of judgment; every 10 ft. should be sufficient if the wall is well tied.

Exterior Bracing for Walls.—When bolts or wire ties are used the bracing, as mentioned above, merely serves to hold the wall in line against the impact of the concrete. If, however, no internal ties can be used, which is often the case when large masses of concrete are to be held—as in bridge abutments, or when only one side of the wall is to be formed, which is the case when a new wall is built against an old one—then the braces must take the whole pressure and must be designed accordingly.

The braces will take the horizontal pressure on the wales. If they can be placed horizontal, that is, in line with the direction of the thrust, the load on the brace will be the same as the horizontal pressure on the wale, and the size of brace required is found from formula (33) for posts or by the allowable bearing stress on the wale, whichever gives the greater value.

It is more usual, however, that the braces are inclined, being held at the bottom by stakes driven in to the ground. When this is the case the thrust on the brace will increase with the inclination of the brace to the horizontal. For angles with the horizontal of 30, 45 and 60 degrees, the thrust on the brace will be approximately 1·25, 1·5, and 2 times the horizontal pressure on the wale.

The higher the wale the greater can be the inclination of the brace, not only to save timber but because the pressure on the wale will be less. If there are three wales the inclination of the braces may be about 30 degrees for the lower one, 45 degrees for the middle one, and 60 degrees for the upper one. They are often held by the same stake.

Horizontal Sheathing.—Sometimes, though not often, wall sheathing is run vertically and the studs horizontally, in which case we have the same conditions as with columns and the forms are designed in the same way, using the formulæ for continuous conditions. In this case the wales will be vertical, but the method of design can be the same as when they are horizontal, assuming that between any two ties each horizontal stud will bring the same load to the wale as the load on the lower stud. This assumption is on the safe side and is sufficiently close, because only low walls are built in this manner and the ties should always be close together.

Wale Bolts.—Knowing the load carried by the wale, the stress in

the bolt and hence its size can be found in the same way as for column yoke bolts.

Usually not heavier than $\frac{3}{4}$-in. diameter bolts are used, so that if the calculated size is greater than this it is better to shorten the span of the wale and use smaller bolts; this is especially true if the bolts are to be pulled afterwards.

CHAPTER IV.

DESIGN TABLES.

Table 1—Slab Sheathing.

In designing sheathing for slabs it will be found that deflection and not strength in bending or shear will govern the thickness. The table is calculated from formula 15, assuming a live load of 75 lbs. per sq. ft.

The timber is assumed to be dressed $\frac{3}{16}$ in. The sheathing is calculated for continuous spans; for single spans deduct one-eighth from the spans given in the table.

One-inch sheathing is the size most ordinarily used for floor slabs. To use the sheathing to the best advantage the joists should be spaced at the maximum span of the sheathing, to which may be added the thickness of the joist, but if this means using an extra heavy joist the spacing will be governed by the strength of the joist.

Table 2—Wall Sheathing.

In this table strength in bending governs the maximum spans. This is because the loads are heavier and the spans shorter than for slabs.

For walls up to 10 ft. high the weight of the equivalent fluid causing horizontal pressure is taken as 125 lbs. per cu. ft., for walls 10 ft. to 20 ft. high as 100 lbs. per cu. ft., and for walls higher than 20 ft. as 75 lbs. per cu. ft.

The table is calculated from formula 46, and the deflection will be less than $\frac{1}{8}$ in. by formula 47.

It is usually possible to use wall sheathing at its maximum span, this being calculated at the bottom of the wall, and so studs should be spaced at the spans given in the table, to which may be added the thickness of the stud.

One-inch or $1\frac{1}{4}$-in. sheathing is usually used for walls.

Table 3—Column Sheathing.

As for wall sheathing, strength in bending will govern the span.

The pressure is assumed to be constant between yokes, with intensity equal to that at the lower yoke, and the formula for calculating the sheathing will be the same as for wall sheathing, "w" being taken at 125 for all heights.

The spans will be the same as for wall sheathing for heights up to

TABLE I.

MAXIMUM SPAN OF FLOOR SHEATHING FOR VARIOUS THICKNESSES OF SLAB

Thickness of Slab	Thickness of Sheathing 1"	1¼"	1½"	2"
3"	32½ ins.	39½ ins.	46 ins.	59 ins.
4"	31½	38	45	57½
5"	31	37½	44	56
6"	30	36½	43	55
7"	29½	36	42	54
8"	29	35	41½	53
9"	28½	34½	41	52
10"	28	34	40	51½
11"	27½	33½	39½	50½
12"	27	33	39	50

Live Load = 75 lbs. sq. ft.

$M = \frac{wl^2}{10}$

$l = 10{\cdot}25\sqrt[4]{d^3/w}$

$D = \frac{1}{8}"$

TABLE. 2.

MAXIMUM SPAN OF WALL SHEATHING FOR VARIOUS HEIGHTS OF WALL

h/d	1"	1¼"	1½"	2"
6 ft.	16 ins.	21 ins.	25½ ins.	35½ ins.
8	14	18	22	31
10	12½	17	21	28
12	12	16	20	27½
14	11½	15	18½	26
16	11	14	17½	24
18	10	13½	16½	23
20	10	13½	16	23
22	10	13½	16	23
24	10	13½	16	23
26	9½	13	16	22
28	9½	12½	15½	21
30	9	12	15	20½

$l = \sqrt{\frac{166{\cdot}67\, bd^2}{wh}}$

$D < \frac{1}{8}''$

$M = wh\,l^2/10$

$h < 10',\ w = 125$

$h,\ 10'\ \text{to}\ 20',\ w = 100$

$h > 20',\ w = 75$

TABLE. 3.

MAXIMUM SPAN OF COLUMN SHEATHING FOR VARIOUS HEIGHTS OF COLUMN

$$l = \sqrt{\frac{166.67\,bd^2}{wh}}$$

$$D < 1/8''$$

$$M = whl^2/10$$

h/d	1"	1¼"	1½"	2"
6 ft.	16 ins	21 ins	25½ ins.	33½ ins
8	14	18	22	31
10	12½	16	20	27½
12	11	14½	18	25
14	10½	13½	17	23
16	9½	12½	15½	21½
18	9	12	15	20½
20	8½	11½	14	19½
22	8	11	13½	18½
24	8	10½	13	17½
26	7½	10	12½	17
28	7½	9½	12	16½
30	7	9	11½	16

$w = 125$

10 ft., but for greater heights the spans will be less. Deflection will be less than $\frac{1}{8}$ in.

For small-size columns the maximum span of the sheathing will govern the spacing of the yokes.

One-inch and $1\frac{1}{2}$ in. are the thicknesses usually used, the latter when the forms are to be used several times over. Timber is assumed to be dressed $\frac{3}{16}$ in.

Table 4—Joists.

This table is calculated from formulæ 5 and 12, assuming single spans, which is the usual condition in beam and girder construction. When the joists have one or more centre supports, 25 per cent. may be added to the spacings in the table, using a span equal to the distance between supports.

Spacings to the left of the heavy line will give a deflection less than $\frac{1}{8}$ in. with a live load of 75 lbs. per sq. ft., and to the right of the heavy line the spacings will give a deflection less than $\frac{1}{8}$ in. with a live load of 40 lbs. per sq. ft.

Unless heavy construction loads are likely to be placed on the slab before it has set the latter values may be used.

It will be noticed that the maximum span for deflection depends only on the depth and not the width of the joist.

For maximum economy of material choose joists that can be spaced at the maximum span of the sheathing; this, however, will not always give the cheapest design, since the larger size joists will cost more than the smaller for the same volume of timber, and it is often cheaper to use smaller joists spaced closer together than required by the strength of the sheathing.

Spacing for shear is calculated by formula 9; it will always be greater than that allowable for bending if the full size of joist is used. If, however, the depth of the joist is decreased at the support, as is sometimes necessary for curved slabs and arch rings, then the unit shearing stress should be investigated.

The thickness of the ledger carrying the joists on the beam side is given by formula 23; it will vary from 1 in. to 2 ins., and this will govern the size of the ledger that is nailed on to the beam side.

All joists are assumed to be dressed $\frac{1}{4}$ in. on each dimension. For undressed timber of full size, multiply the spacings given by the following factors:—

For 2 in. by 4 in.	joists	multiply by	1·3
2 in. by 6 in.	,,	,,	1·25
2 in. by 8 in.	,,	,,	1·20
2 in. by 10 in.	,,	,,	1·20
3 in. by 4 in.	,,	,,	1·25
3 in. by 6 in.	,,	,,	1·20
3 in. by 8 in.	,,	,,	1·15
3 in. by 10 in.	,,	,,	1·15

For 4 in. by 6 in. joists	multiply by	1·15
4 in. by 8 in.	,, ,,	1·15
4 in. by 10 in.	,, ,,	1·125

The commonest size used for joists is 2 in. by 6 in.

Table 5—Ledgers and Wales.

This table gives the size and span of ledgers supporting joists, or wales supporting studs, for various spacing of joists and studs. They are assumed to be continuous over two spans or more.

The concentrated load brought by the joist or stud is first calculated and the nearest to this load is then found in the table, which will give the size and maximum span of the supporting member. This maximum span governs the spacing of the posts.

Generally, several alternate sizes and spans can be used, so it is necessary to make alternate designs to determine which is the most economical

This table is calculated from formula 25. The live load can be assumed at 75 or 40 lbs. per sq. ft. as desired; generally the latter is sufficient.

Shear and deflection can be investigated by formulæ 26 to 32. For shear one joist should be assumed to be close to the support.

The width of the member is also important, as the allowable bearing stress on the post should not be exceeded (see formula 23).

If the post is not square the long side should be placed parallel to the ledger.

Two small sizes are often doubled up to make a ledger or wale, using the same size as for joists and studs, thus keeping the number of different sizes to a minimum and also saving the extra cost of the larger sizes.

The ordinary sizes used are 3 in. by 4 in. or 6 in. to 4 in. by 4 in. or 6 in., either singly or by doubling up.

Tables 6 to 9—Column Yokes.

These tables give the required spacing of column yokes for various sizes of yokes and heights and dimensions of columns. They are calculated from formulæ 37 and 41. Strength in bending will govern the spacing; except where noted the deflection will be less than ⅛ in. Shearing stress will be safe. It is assumed that the span of the yokes is 1 ft. greater than the dimension of the column.

For the short side of a rectangular column or the side of a square column that does not carry the bolts, the span will be equal to or less than the side of the column; if desired the yoke can therefore be made smaller, as noted, but the spacing will be the same.

It will be noted that in some cases two sets of spacing are given to a column; this is when the strength of the sheathing governs the spacing. Where only one set of spacings is given either of the thicknesses of the sheathing given, or greater, can be used.

Yokes are not often spaced closer than 9 in. at the bottom; when a

TABLE 4.

TABLE. 4.

MAXIMUM SPACING OF FLOOR JOISTS FOR VARIOUS SPANS AND THICKNESSES OF SLAB

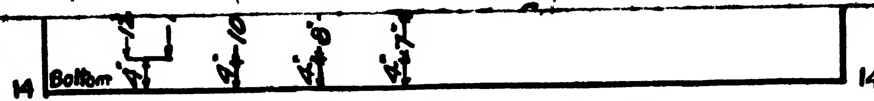
$S = \frac{160}{w'}$ Figures to left

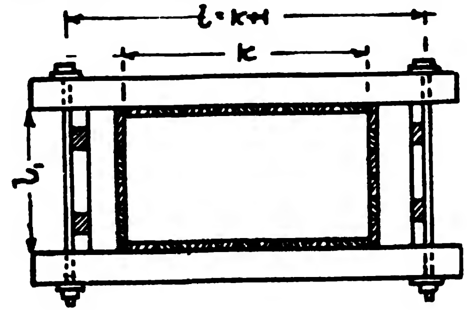
	3"x8"			35	30	26	23	20	17	15	14	12
	3"x10"						36	31	28	25	22	20
	4"x6"	38	31	26	22	19	17	15	13	11	10	
	4"x8"					35	30	27	24	21	19	17
	4"x10"								38	34	32	29
12"	2"x6"	17	14	11								
	2"x8"	30	25	21	18	15	13	12	10			
	2"x10"		40	33	28	24	21	19	16	15	13	12
	3"x6"	26	22	18	15	13	11	10				
	3"x8"		40	33	28	24	21	19	16	15	13	12
	3"x10"					39	34	29	26	23	21	19
	4"x6"	36	30	25	21	18	16	14	12	11	10	
	4"x8"				39	33	29	25	22	20	18	16
	4"x10"								36	32	29	26

TABLE 5.

	36	530	970	1540	840	1530	2410	1140	2090	3290	1760	3200	5050
	39	600	1090	1730	950	1720	2720	1290	2350	3710	1980	3600	5680
	42	690	1250	1970	1080	1960	3110	1470	2680	4230	2260	4100	6480
7'-6"	12	170	310	480	260	480	760	360	660	1040	550	1010	1590
	15	210	390	610	330	610	960	460	830	1310	700	1280	2020
	18	250	450	710	390	700	1120	530	960	1520	810	1470	2330
	21	290	530	840	460	830	1320	620	1130	1790	960	1740	2750
	24	330	600	950	520	950	1500	710	1290	2040	1090	1980	3130
	27	360	650	1020	560	1020	1610	760	1390	2190	1170	2130	3360
	30	390	700	1100	600	1100	1740	820	1500	2370	1270	2300	3620
	33	420	760	1200	660	1190	1890	900	1630	2570	1380	2410	3940
	36	460	830	1310	720	1310	2070	980	1780	2820	1510	2740	4320
	39	510	920	1450	800	1440	2290	1080	1970	3110	1670	3030	4710
	42	570	1030	1630	890	1620	2560	1210	2210	3480	1860	3380	5340
8'-0"	12	150	270	430	230	430	680	320	580	920	490	900	1420
	15	180	330	530	290	530	840	400	720	1140	610	1110	1740
	18	220	400	620	340	620	990	470	850	1340	720	1300	2060
	21	250	460	730	400	720	1150	540	990	1560	830	1510	2390
	24	300	550	860	470	860	1360	640	1170	1850	990	1800	2830
	27	320	580	920	500	920	1450	690	1250	1970	1060	1920	3030
	30	340	620	980	540	980	1560	740	1340	2110	1130	2050	3240
	33	370	670	1060	580	1060	1680	800	1440	2280	1220	2210	3490
	36	400	730	1150	630	1150	1820	860	1560	2470	1320	2400	3780
	39	440	800	1260	690	1250	1980	940	1700	2690	1440	2620	4120
	42	480	870	1380	760	1370	2180	1030	1870	2960	1580	2880	4540

TABLE 6.





For side l_1

$$S = \frac{17.06\,bd^2}{h\,l_1^2}$$

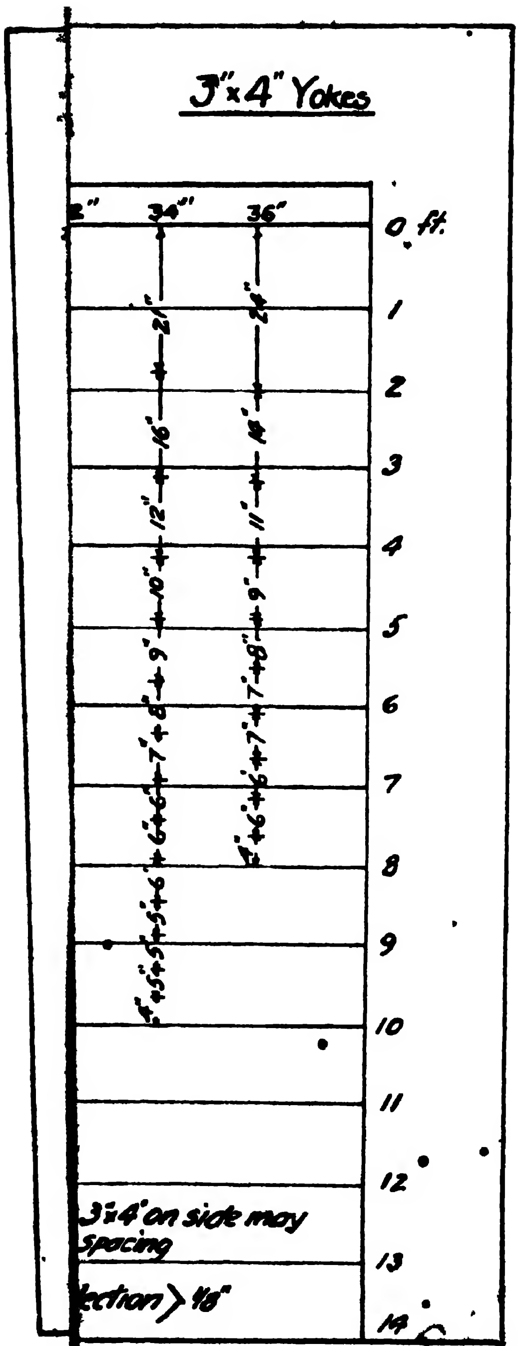
$$d = \cdot 32\,l_1^2$$

Yoke Formulae :-

Bending, $S = \dfrac{17.06\,bd^2}{h(l^2-1)}$.

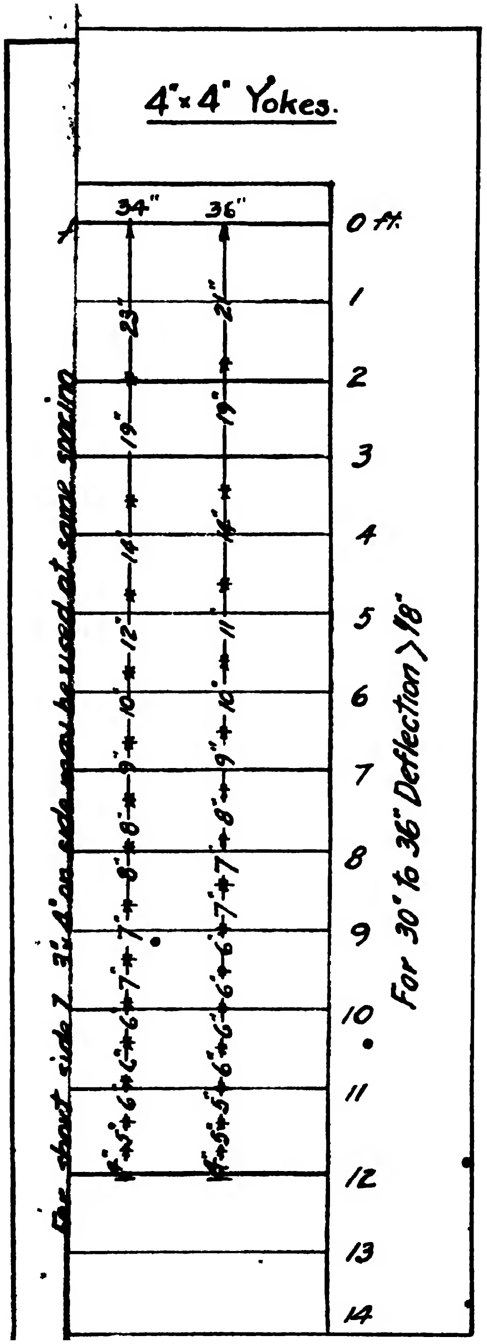
Deflection, $D = 1/8''$, $d = \dfrac{\cdot 32\,l^4}{l^2 \cdot 1}$

TABLE 7.



3"x4" Yokes
34"
36"
0 ft.
1
2
3
4
5
6
7
8
9
10
11
12
13
14
3"x4" on side may
spacing
lection) 48"

TABLE 8.



4"×4" Yokes.
34"
36"
0 ft.
1
2
3
4
5
6
7
8
9
10
11
12
13
14
3"×4" on side may be used at same spacing
For 30" to 36" Deflection > 18"

TABLE 9.

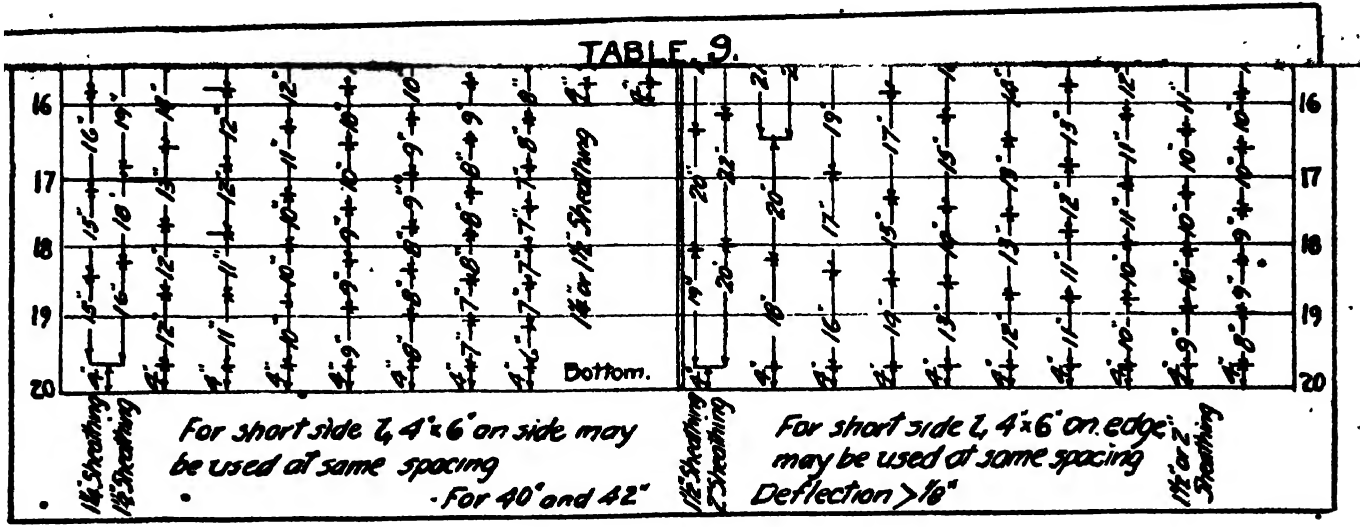


TABLE 9.
16
17
18
19
20
Bottom.
1¼" or 1½" Sheathing
1¼" Sheathing
1½" Sheathing
For short side L, 4"x6" on side may be used at same spacing
For 40' and 42"
1½" Sheathing
2" Sheathing
For short side L, 4"x6" on edge may be used at same spacing
Deflection > 1/8"
1½" or 2" Sheathing

TABLE 10.

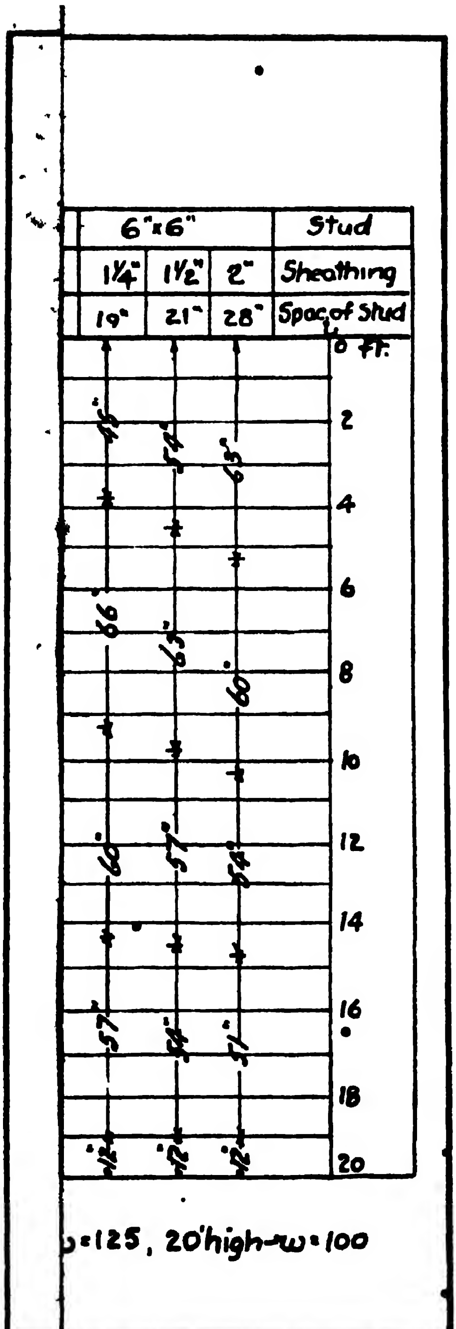

6"x6"
Stud
1¼"
1½"
2"
Sheathing
19"
21"
28"
Spac. of Stud
0 ft.
2
4
6
8
10
12
14
16
18
20
45
54
63
66
63
60
60
57
54
57
54
51
=125, 20' high—w=100

TABLE. 11.
SAFE LOADS ON POSTS

Size of Post	3"x4"	4"x4"	4"x6"	6"x6"
Bearing @ 400	4100 lbs.	3600 lbs	8600 lbs.	13300 lbs
" " 600	6200	8400	12900	20000
Height 4'	8000	11800	18100	29600
6'	7000	10700	16400	27900
8'	5800	9500	14600	26100
10'	4700	8400	12900	24400
12'	3600	7300	11200	22700
14'	2400	6200	9500	21000
16'	1300	5100	7800	19300
18'	180	3900	6000	17600
20'	—	2800	4300	15900

$W = 1000(1 - l/80d)bd$

For Bearing:-

$W = 400.b.d.$

or $= 600\,bd$

closer spacing is required it is better to use a heavier yoke or to put a centre bolt through the column and so reduce the span. When a centre bolt is used the same spacing should be used as for a column, of half the dimension of the side.

Columns are poured to the underside of the deepest members framing into them, so that the height to take for the column will be the height from the bottom of the deepest beam to the floor or footing below.

For any height the first yoke from the bottom should be placed about 4 in. up, and the spacing above should start with the distance cut by a horizontal line through the height of the column; or, if this distance is cut at about the centre, start with the average between this and the distance next above. For instance, for a column 9 ft. high and 18 in. side, place the first yoke 4 in. up and the spacing above will be 15 in., 16 in., 18 in., etc., with 3 in. by 4 in. yokes.

It is common, though not good practice, to space the yokes equally all the way up at, say, 14 in. to 16 in. on centre for ordinary size columns and heights in buildings. If this spacing is suitable for the lower yokes it will mean only a waste of material above, but if a smallers pacing should be used at the bottom the lower yokes are liable to bulge, and this is often the case in practice when equal spacing is used.

It is often claimed that it is easier for a carpenter to lay out equal spaces. This may be so, though it is doubtful; the saving of a yoke per column will, however, more than offset any possible extra labour cost in laying out the yokes correctly.

When column forms are to be used over and over again on successive floors, with varying heights and sizes of column, the spacing of the yokes in the first place should be such that they are on the safe side for the worst conditions.

For column forms to be used only once or twice 1 in. sheathing is sufficient, but if the forms are to be used several times over it is better to use 1½ in. sheathing. Sheathing thicker than 1½ in. is seldom used except for unusual conditions.

The tables are for dressed timber; for rough timber the spacings given may be multiplied by the same factors as given for joists, except where the spacing is governed by the safe span of the sheathing.

The sizes most used are 3 in. by 4 in. or 4 in. by 4 in.

Table 10—Wall Ties and Wales.

The spacing of studs is governed by the maximum span of the sheathing from Table 2, to which may be added the width of the stud.

If the studs are simply tied through the wall with no wales, the table will give the maximum spacing for the ties. If wales are used, as is customary, the table gives the spacing of the wales. It is calculated from formulæ 49 and 50, the deflection governing the spacing and the pressure being assumed constant between any wale and the one above it, with intensity equal to that at the lower one.

When wales are used they need not be tied through the wall at every

stud, but the maximum span between ties can be found from Table 5 for the size it is desired to use. Care must be taken to see that all the studs have a bearing on the wale, wedging them if necessary when a stud is a little under size.

For any height of wall place the first wale or tie about 12 in. from the bottom and start the spacing from that point with the distance cut by a horizontal line through the height of the column.

Wall forms more than 12 ft. or 14 ft. high are seldom built, as it is usually cheaper to build the forms for half the height and raise them. If the forms are built to the top for high walls, they are generally poured in two lifts. In either of these cases, therefore, the spacing of the wales need only be that for a wall of half the total height. In some cases for strength and watertightness a high wall has to be poured without a break, in which case the forms must be designed accordingly.

It must be remembered that the strength of wall forms will depend on the tie wires or bolts, and the tables will give the maximum spacing of these. They should not be used too sparingly; additional ones should be added at weak points and corners, and outside braces also used.

For low walls, 2 in. by 4 in. studs and 3 in. or 4 in. by 4 in. wales can be used; for walls over 10 ft., 3 in. or 4 in. by 4 in. studs and 4 in. or 6 in. by 6 in. wales may be used.

One-inch or 1½-in. sheathing will do for any except heavy retaining walls.

Wales are often made by doubling up smaller sizes with the bolts placed between them.

Table 11—Posts.

This table is calculated from formula 33. Posts over 8 ft. in height should be braced in both directions at their centre, usually with 1 in. by 6 in. timber, and the height then for calculating the allowable load will be half the total height of the post.

For very high posts the bracing should be at the third points.

If the bottom of the timber carried by the post is wider than the post, the size of the post is governed by the bearing stress on the timber, either 400 or 600 lbs. per sq. in. according to the quality of the timber or whether hardwood caps or steel bearing plates are used.

It will be seen that theoretically the loads depending upon bearing stress are equivalent to those for columns 10 ft. high and over. Nevertheless, bracing should always be used. The formula for posts assumes no eccentricity of loading and hence no bending, but in formwork some bending action will nearly always be present, chiefly because the posts are seldom placed absolutely vertical and often several inches out of plumb, in which case a much smaller load can safely be placed on the post. Bracing by reducing the unsupported height will take care of this bending action.

Wedges, preferably hardwood, should always be placed beneath the posts to allow for adjustment in height.

For rough posts, add 10 per cent. to all loads given in the table.

For ordinary building work, 3 in. or 4 in. by 4 in. posts are used, and for heavier work 6 in. by 6 in. It is a common rule amongst superintendents that a 3 in. or 4 in. by 4 in. post will carry a cu. yd. of concrete, or 4000 lbs.

Beam Bottoms.

A table is not necessary for the design of beam bottoms, since the allowable span does not vary greatly with the size of the beam. They are designed as continuous shallow beams supporting a uniform load and spanning between the supporting posts. Formulæ 3 and 14 apply, "*w*" being the weight of the concrete beam and the live load on the beam. It is assumed that none of the slab load is carried by the beam bottom, and that the beam bottom is independent of, and therefore receives no support from, the side forms.

For a deflection of $\frac{1}{8}$ in., deflection will govern the allowable span between posts.

Two-inch dressed timber is almost invariably used for beam bottoms. The maximum span is independent of the width of the beam and varies only slightly with the depth, the depth being measured from the top of the slab.

With 2 in. dressed timber, a deflection of $\frac{1}{8}$ in., and a live load of 40 lbs. per sq. ft, the following rule can be used :—

For depth of beam	up to 16 in.,	space posts	4 ft. 3 in.	apart.
,,	from 16 in. to 24 in.	,,	4 ft. 0 in.	,,
,,	,, 24 in. to 36 in.	,,	3 ft. 9 in.	,,
,,	,, 36 in. to 48 in.	,,	3 ft. 6 in.	,,

Posts supporting beam bottoms should always have a cross-piece at the top, braced back to the post, in order to give the bottom full bearing.

Application of Tables.

We now have all the data necessary quickly to design any problem in formwork or to check the strength of formwork already erected.

Sizes should not be picked from the tables at random without considering the two important points—timber available and relative cost of different sizes.

It is not economical to hold up work waiting for a certain size timber when another size that can be delivered immediately can be used equally as well.

Also, before deciding on a design the question of how many times over it will be necessary to use the forms must be considered. Light forms will not stand much re-handling, and in a multiple-story building it will pay to put more timber into the built-up units of the first floor than is required for strength, because less patching and renewals will be required on successive floors.

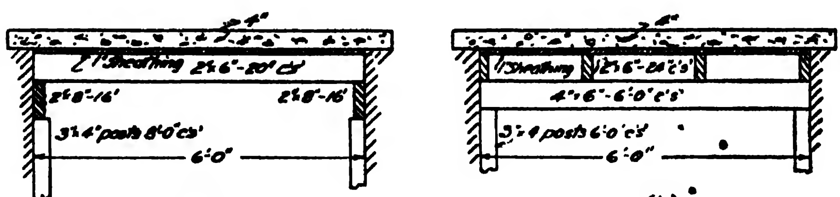
For almost any condition several different designs can be made, all having the same strength, and it is necessary to make comparisons of the relative economy unless there is no choice in the sizes of members to use.

CHAPTER V.

DESIGN PROBLEMS.

Design 1 : Simple Slab.

Design forms for a 4-in. corridor slab, span 6 ft., supported on brick walls.



(a) (b)

DESIGN-1

(*a*) **Joists Spanning across Slab.**—From Table 1, 1-in. sheathing will span up to 31½ in.; from Table 2, 2 in. by 6 in. joists at 20 in. on centre can be used; so strength of joists will govern spacing.

The joists will be supported at their ends by ledgers along the wall. The load carried by each joist to ledger will be $(48 + 40)\ 1{\cdot}67 \times 3 = 440$ lbs. From Table 5, we have the choice of 2 in. by 6 in. spanning 6 ft., 2 in. by 8 in. spanning 8 ft., or 3 in. by 6 in. spanning 7 ft. 6 in., or 4 in. by 6 in. spanning 8 ft. Assuming 3 in. by 4 in. posts, the 2 in. by 8 in. will give the least amount of timber and will be used.

Maximum load on post (calculated from formula 26) with one joist over post = 2130 lbs. The 4-in. side of the post will be placed against the wall. Bearing required for ledger $= \dfrac{2130}{400 \times 1{\cdot}75} = 3$ in., and we have 3¾ in. From Table 11 it will be seen that a 3 in. by 4 in. post is sufficiently strong.

(*b*) **Longitudinal Joists.**—If it is desired to run the joists longitudinally they can be designed as continuous and the spacing of the joists may be increased 25 per cent. With one centre joist the span of the sheathing would be 36 in., which is too great for 1-in. sheathing, so two centre joists will be used. The spacing will then be 24 in., and from Table 4 for 2 in. by 6 in. joists and 4-in. slab, adding 25 per cent. to spacing, the span or distance between ledgers can be 6 ft.

Load from each joist on ledger $= 88 \times 2 \times 6 = 1056$ lbs., and from

Table 5, for a span of 6 ft. and joists 24 in. centre to centre a 4 in. by 6 in., made up of two 2 in. by 6 in., can just be used.

Load on post will be $1056 \times 1{\cdot}5 = 1584$ lbs. This being less than before no further calculations are necessary.

Comparing the relative economy in timber, we have for one lin. ft. of slab in cu. ft. of timber, assuming posts 10 ft. long:

	(a)		(b)	
Sheathing	$\frac{1}{12} \times 6$	$= 0{\cdot}5$		$= 0{\cdot}5$
Joists	$\frac{12 \times 6 \times 12}{144 \times 20}$	$= 0{\cdot}3$	$\frac{12 \times 4}{144}$	$= 0{\cdot}333$
Ledgers	$\frac{16 \times 2}{144}$	$= 0{\cdot}222$	$\frac{24 \times 6}{144 \times 6}$	$= 0{\cdot}167$
Posts	$\frac{12 \times 10 \times 2}{144 \times 8}$	$= 0{\cdot}208$	$\frac{12 \times 10 \times 2}{144 \times 6}$	$= 0{\cdot}278$
		$= 1{\cdot}230$ c.f.		$= 1{\cdot}278$ c.f.

It will be seen that there is little difference in the amount of timber required in the two designs. The second design would be slightly better perhaps because only one size of timber is required and the joists can be used in long lengths from which there would be better salvage than if they were cut up into 6 ft. lengths, while cutting the sheathing would not matter much since short lengths can always be used.

Design 2: Simple Slab, Long Span.

Design forms for a 7 in. roof slab 14 ft. by 20 ft., 10 ft. high.

A span of 14 ft. will require one or two rows of posts to reduce the span.

(*a*) **One Row of Posts.**—Joists will be continuous, so 25 per cent. can be added to the spacing. From Table 4, for 7 ft. span and 7 in. slab, 2 in. by 8 in. joists at $21 \times 1{\cdot}25 = 26$ in. c.s. can be used. From Table 1, 1-in. sheathing will span up to $29\frac{1}{2}$ in.

Load from each joist on ledger $= (84 + 40) \times 2{\cdot}17 \times 7 = 1885$ lbs.

From Table 5 we have choice of 2 in. by 8 in., span 4 ft. 6 in.; or 4 in. by 8 in., span 6 ft. The former will give less timber per lin. ft., so will be used.

Maximum load on post with one joist over post $= 1885\ (1 + 2\ ({\cdot}52 + {\cdot}037)) = 4000$ lbs.

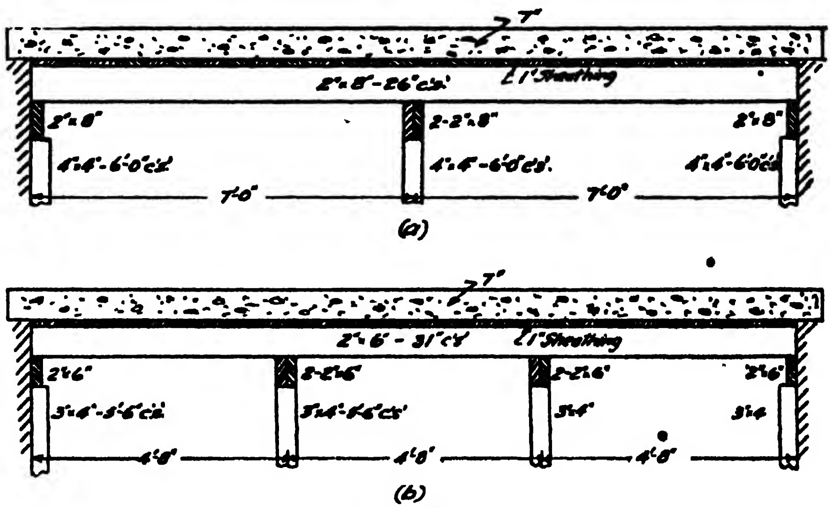
Bearing stress on ledger $= \dfrac{4000}{1{\cdot}75 \times 3{\cdot}75} = 600$ lbs. sq. in., which is too high and indicates the ledger is too narrow, so a 4 in. by 8 in., made up of two 2 in. by 8 in., must be used.

Load on post now $= 1885(1 + 2(\cdot 64 + \cdot 28)) = 5350$ lbs., and bearing stress $= \dfrac{5350}{3\cdot 75 \times 3\cdot 75} = 380$ lbs. sq. in., which is safe.

Load on wall ledgers = half the above = 942 lbs.

From Table 5 one 2 in. by 8 in. will carry the load at 6 ft. span.

Wall posts will be the same as interior posts for convenience. Bearing stress on wall ledger $= \dfrac{2675}{1\cdot 75 \times 3\cdot 75} = 407$ lbs. sq. in., or just at the limit of the allowable stress.



DESIGN-2

(*b*) **Two Rows of Posts.**—If it is desired to use smaller joists and ledgers, use two rows of posts. Span now is 4 ft. 8 in. In Table 4 interpolating between 4 ft. 6 in. and 5 ft. spans, 2 in. by 6 in. at $27 \times 1\cdot 25 = 34$ in. can be used. But from Table 1 maximum span of 1-in. sheathing $= 29\frac{1}{2}$ in. and adding $1\frac{1}{2}$ in. for width of joist gives 31 in. for maximum spacing of joists.

Loads on ledger $= 124 \times 2\cdot 58 \times 4\cdot 67 = 1500$ lbs.

From Table 5, using same size timber, 4 in. by 6 in. (two 2 in. by 6 in.) will carry the loads at 5 ft. 6 in. span.

Load on post $= 1500(1 + 2(\cdot 53 + \cdot 06)) = 3270$ lbs. From Table 10 a 3 in. by 4 in. post will do, checking bearing stress,

$$\text{stress} = \frac{3270}{3\cdot 75 \times 2\cdot 75} = 318.$$

Loads on wall ledger = 750, and one 2 in. by 6 in. will carry the loads at the same span of 5 ft. 6 in.

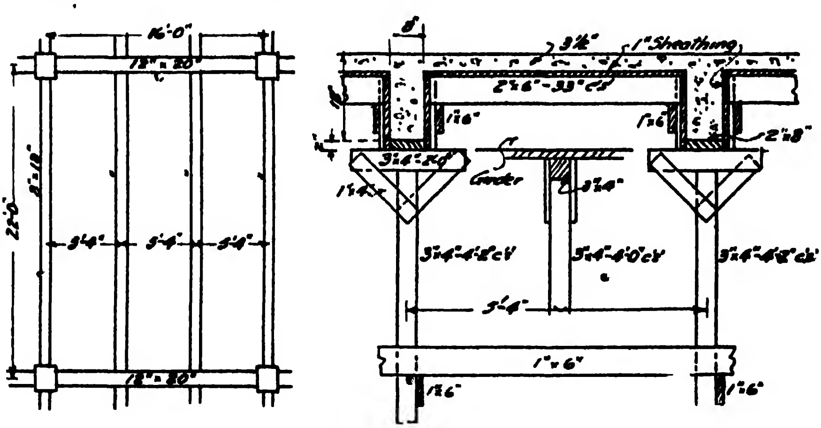
Comparing the two designs for economy of material, in cu. ft. of timber per lin. ft. of slab, assuming 1 in. × 6 in. post braces each way.

		(a)		(b)	
Sheathing		$\frac{14}{12}$	$= 1\cdot17$	$\frac{14}{12}$	$= 1\cdot17$
Joists		$\frac{16 \times 12 \times 14}{144 \times 26}$	$= 0\cdot72$	$\frac{12 \times 12 \times 14}{144 \times 31}$	$= 0\cdot45$
Ledgers		$\frac{64}{144}$	$= 0\cdot45$	$\frac{72}{144}$	$= 0\cdot50$
Posts		$\frac{16 \times 30}{144 \times 6}$	$= 0\cdot56$	$\frac{12 \times 40}{144 \times 5\cdot5}$	$= 0\cdot61$
Braces		$\frac{6 \times 32}{144 \times 6}$	$= 0\cdot22$	$\frac{6 \times 36}{144 \times 5\cdot5}$	$= 0\cdot27$
			$3\cdot12$ c.f.		$3\cdot00$ c.f.

There is not much difference between the two designs for amount of material used, so it will be a question of the relative cost of sizes.

Design 3: Beam and Girder Panel, Short Span Slab.

Design forms for typical bay of beam and girder construction, consisting of 3½-in. slab, 8 in by 18 in beams, span 22 ft. and 5 ft 4 in. on centres and 12 in. by 20 in. girders, span 16 ft



DESIGN-3

From Table 1, 1-in. sheathing for 3½-in. slab will span 32 in.

Span of joists will be 4 ft. 6 in. From Table 4, interpolating between 3 in. and 4 in. slabs, we can use 2 in. by 4 in. at 16½ in. on centre, or 2 in. by 6 in. at 39 in. on centre, which, however, must be reduced to $32 + 1 = 33$ in. on centre because of the deflection of the sheathing. The latter will be more economical, so will be chosen.

Load carried to ledger, including 40 lbs. live load $= (42 + 40) \times 2\cdot75 \times 2\cdot25 = 510$ lbs.

Width of ledger required for bearing $= \dfrac{510}{400 \times 1{\cdot}75} = {\cdot}73$ in., so 1 in. by 6 in. can be used.

Beam bottoms will be 2 in. by 8 in., and according to our rule a 16-in. depth requires posts 4 ft. 3 in. on centre and a 24 in. depth 4 ft. on centre; since the clear span of the beams is 21 ft., it will be satisfactory to space the posts 4 ft. 2 in. on centre.

The girder bottom will be 2 in. by 12 in., made out of two 2 in. by 6 in. The spacing of the posts under the girders may be 4 ft.

Load on beam posts

$$= (82 \times 5{\cdot}33 \times 4{\cdot}17) + (8 \times 14{\cdot}5 \times 4{\cdot}17) = 2305 \text{ lbs.}$$

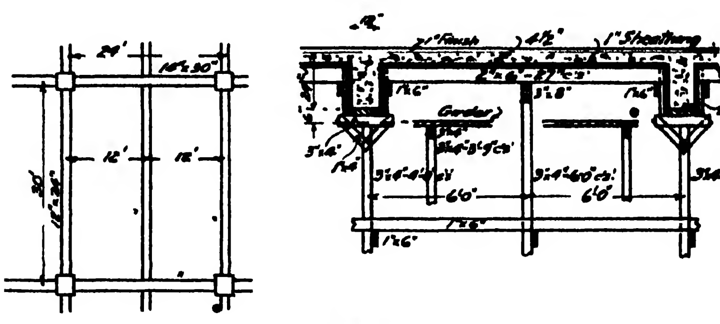
Load on girder posts

$$= (82 \times 5{\cdot}17 \times 4) + (12 \times 16{\cdot}5 \times 4) + (8 \times 14{\cdot}5 \times 4{\cdot}17) = 2975 \text{ lbs}$$

From Table 11, 3 in. by 4 in posts can be used.

Design 4: Beam and Girder Panel, Long Span Slab.

Design forms for typical beam and girder panel consisting of 4½ in. slab with 1 in. finish of cement, with 12 in. by 24 in. beams spanning 30 ft and 14 in. by 30 in girders spanning 24 ft.



DESIGN-4

Total thickness of slab will be 5½ in. From Table 1, 1-in sheathing will span 30½ in. Clear span of joists is 11 ft. To span this without a centre support would mean using heavy joists close together, so one centre ledger will be used, reducing the span to 5 ft. 6 in., and joists will be continuous, so that 25 per cent. can be added to the spacing.

Interpolating between 5 in. and 6 in. slabs, and adding 25 per cent., we can use 2 in. by 6 in. at 27 in. on centre.

Loads on centre ledger $= (66 + 40)2{\cdot}25 \times 5{\cdot}5 = 1315$ lbs. From Table 5 we have choice of 2 in. by 8 in. span 5 ft., 4 in. by 6 in. span 5 ft. 6 in., 3 in. by 8 in. span 6 ft., 4 in. by 8 in. span 7 ft. 6 in. For economy in material they are approximately in the ratio of $1 : 1{\cdot}1 : 1{\cdot}05 : 1{\cdot}1$. Available material and relative cost will decide; we will use 3 in. by 8 in. span 6 ft.

Loads on beam ledger will be 658 lbs., and we can tell from previous design that a 1 in. by 6 in. will be sufficient.

Maximum load on ledger posts $= 1315(1 + 2(\cdot 625 + \cdot 25)) = 3025$ lbs.

Bearing stress on 4 in. post $= 3025/3\cdot 75 \times 2\cdot 75 = 293$, so we can use 3 in. by 4 in. posts.

To show the use of the formulæ we will investigate shear and deflection of the ledger.

With one joist 4 in. from centre of post

$$V = 1315\frac{(5\cdot 67 + 3\cdot 42 + 1\cdot 17)}{6} = 2250$$

and unit shearing stress $v = \dfrac{1\cdot 5 \times 2250}{2\cdot 75 \times 7\cdot 75} = 159$ lbs. sq. in., which is well within the allowable.

Deflection (formula 32),

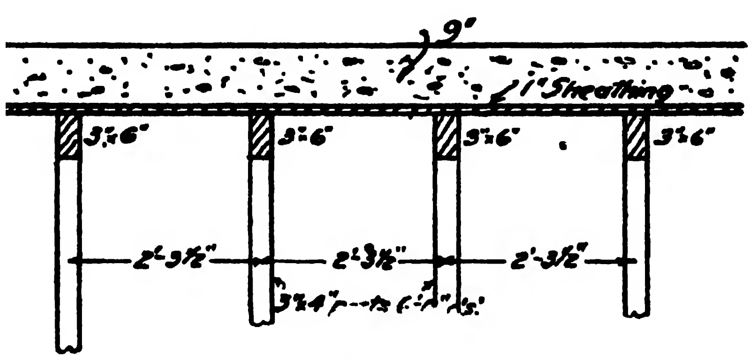
$$D = \frac{5 \times 1315 \times 12}{384 \times 1{,}200{,}000 \times 2\cdot 75 \times 7\cdot 75^3} \times (72^3 + 2 \times 9(3 \times 72^2 - 4 \times 9^2))$$

$= 0\cdot 087$ in. which is less than $\frac{1}{8}$ in.

Beam bottoms will be 2 in. thick, and the spacing of the posts 4 ft. Girder bottoms will be 2 in. thick, and the spacing of the posts will be 3 ft. 9 in.

Design 5: Mushroom Slab, One-Way Design.

Design forms to support a 9 in. flat floor slab without beams; floor height, 11 ft.



DESIGN 5

This design can be used for any large area of floor without beams.

From Table 1, 1 in. sheathing will span 28½ in.

From Table 4, continuous joists 3 in. by 6 in. spanning 6 ft. can be placed 27½ in. on centre.

Load on posts $= (108 + 40) \times 2\cdot 29 \times 6 = 2030$ lbs. Use 3 in. by 4 in. posts.

Design 6: Mushroom Slab, Two-Way Design.

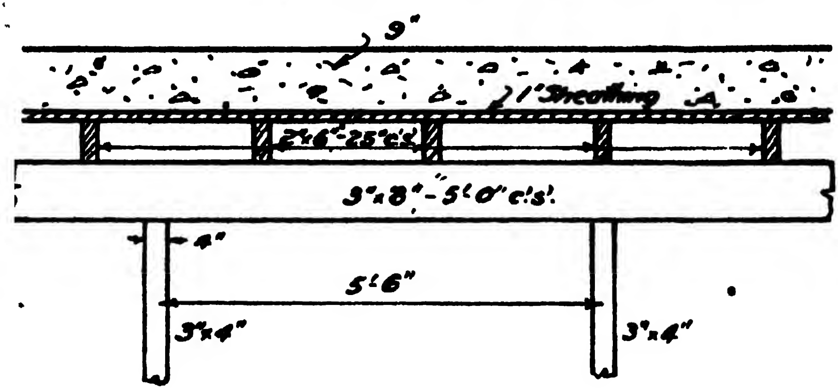
Design forms for a 9 in. slab as before, using joists and ledgers.

Instead of carrying the joists individually on posts, ledgers may be used with smaller joists.

From Table 4 (continuous joists), 2 in. by 6 in. at 25 in. on centre will span 5 ft.

Loads on ledgers from joists = 148 × 2·08 × 5 = 1540 lbs.

From Table 5, the best ledgers to use are 4 in. by 6 in. span 5 ft.,



DESIGN 6

or 3 in. by 8 in. span 5 ft. 6 in.; we will use the latter as it takes less timber.

Maximum load on posts = 1540 × (1 + 2(·621 + ·243)) = 4200 lbs.

Bearing stress on post 4 in. wide $= \dfrac{4200}{3{\cdot}75 \times 2{\cdot}75} = 407$, so that it is just possible to use a 3 in. by 4 in. post, with the long side parallel to the ledger.

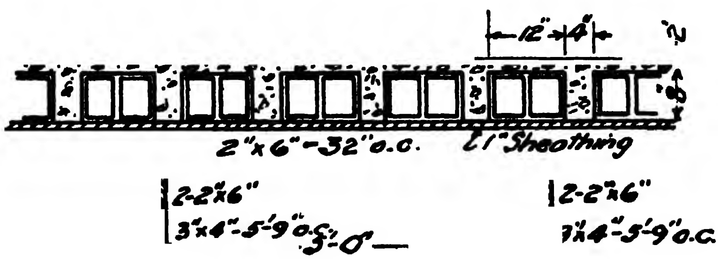
Comparing the two designs 5 and 6 for economy in timber, in cu. ft. of timber per sq. ft. of floor,

(5) Sheathing		= 0·083 c.f.	(6)		= 0·083
Joists	$\dfrac{18 \times 12}{144 \times 27{\cdot}5}$	= 0·055		$\dfrac{12 \times 12}{144 \times 25}$	= 0·040
Posts	$\dfrac{12 \times 10}{144 \times 2{\cdot}29 \times 6}$	= 0·061		$\dfrac{120 \times 1}{144 \times 5{\cdot}5 \times 5}$	= 0·030
Braces	$\dfrac{6 \times 8{\cdot}29}{144 \times 2{\cdot}29 \times 6}$	= 0·025		$\dfrac{6 \times 10{\cdot}5}{144 \times 27{\cdot}5}$	= 0·016
			Ledger	$\dfrac{24 \times 1}{144 \times 5{\cdot}5}$	= 0·030
		0·224 c.f.			= 0·199 c.f.

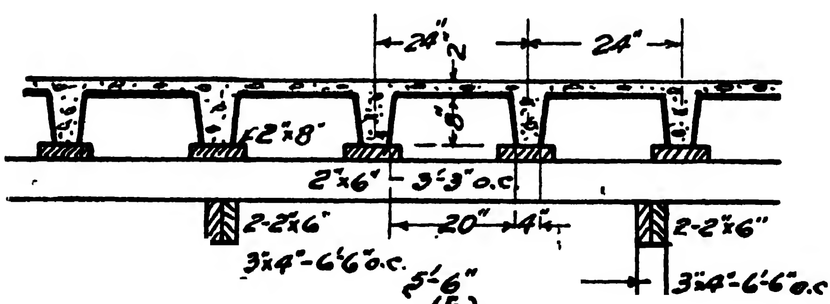
This shows that it is more economical to use the two-way system, saving about 10 per cent. of the timber required by the one-way. This holds good in general, and for that reason is more popular.

Design 7: Light Weight Rib Floors.

By this is meant that system of floor construction which consists of clay, metal, or gypsum tile fillers, combined with narrow concrete joists of long span, supporting a thin slab; the slab may be omitted.



(a.)



(b.)

DESIGN 7

(*a*) **Closed Deck.**—Clay tile fillers are usually 12 in. wide, combined with 4 in. wide joists of variable depth depending on the span carrying a 2 in. slab.

Since the tile is narrow and the joists close together, with tongue and groove sheathing, the heavier weight of the joist is so distributed that the load may be considered uniform, and the deck is often built solid as for a solid slab.

Design forms for a floor 10 *in. deep, consisting of* 4 *in. joists with* 2 *in. slab and* 12 *in. wide by* 8 *in. deep clay tile fillers.*

An 8 in. by 12 in. by 12 in. tile weighs 30 lbs., the slab 25 lbs., and the joist 40 lbs., or total of 95 lbs. per foot of joist, or a uniform load of $95/1 \cdot 33 = 70$ lbs. per sq. ft., which is equivalent to a $5\frac{1}{2}$ in. solid slab. From Table 1, maximum span of 1 in. sheathing is $30\frac{1}{2}$ in.; adding $1\frac{1}{2}$ in. for joist, joists will be 32 in. on centre.

From Table 4, adding 25 per cent. to spacing, for spacing of 32 in. with $5\frac{1}{2}$ in. slab, 2 in. by 6 in. joists will span 5 ft.

Loads carried by joists to ledger $= (70 + 40) \times 2 \cdot 67 \times 5 = 1470$ lbs.

From Table 5, 4 in. by 6 in. ledgers will span 5 ft. 9 in. with these loads.

Load on posts $= 1470(1 + 2(0 \cdot 535 + 0 \cdot 073)) = 3250$ lbs., and a 3 in. by 4 in. will be sufficient.

(*b*) **Open Deck.**—Metal tiles are generally about 20 in. wide at the bottom and are much lighter than clay tiles, so that the load is concentrated at the joists; these forms are therefore built with a 2 in. plank under the joist, the rest of the deck being left open.

Design forms for a floor consisting of 4 *in. wide joists* 8 *in. deep,* 24 *in. on centre carrying a* 2 *in. slab, with metal tile fillers.*

Tiles will weigh about 3 lbs. each, joist 36 lbs., slab 48 lbs., or, say, a total load of 90 lbs. per foot of joist. Live load per foot $= 75 \times 2 = 150$, or total load of 240 lbs. per foot of joist.

Using 2 in. by 8 in. under joists the maximum span is found from formula 14.

$$\text{for } D = 1/8, \ \frac{1}{8} = \frac{3 \times 340 \times l^4 \times 12^4 \times 12}{384 \times 12 \times 1{,}200{,}000 \times 7 \cdot 5 \times 1 \cdot 75^3} \text{ or } l = 3 \text{ ft. } 3 \text{ in.}$$

Loads on joist $= (90 + 80) \times 3 \cdot 25 = 555$ lbs., and from Table 5 a 2 in. by 6 in. will span 5 ft. 6 in.

If desired to use ledgers at 5 ft. 6 in. on centre instead of shoring each joist a 4 in. by 6 in. will span 6 ft. 6 in. with the load of 1565 lbs. per joist. Use 3 in. by 4 in. posts.

Design 8: Floor Slabs with Small Haunches at the Beams.

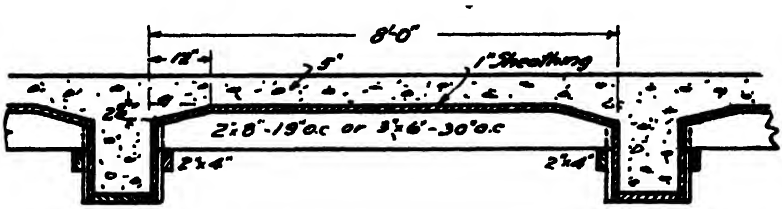
Design forms for a 5 *in. slab, span* 8 *ft. between beams, with haunches* $2\frac{1}{2}$ *in. deep and* 12 *in. long at each beam.*

When the haunch is small the joists can be cut to suit the haunch.

From Table 1, 1 in. sheathing will span 31 in.; from Table 2 we can use 2 in. by 8 in. at 19 in., or 3 in. by 6 in. at 30 in.

As joists will be cut away at the ends the horizontal shearing stress must be investigated, otherwise the design is the same as before.

With 2 in. by 8 in.,



DESIGN 8

$$v = \frac{1{\cdot}5 \times \{(60 + 40) \times 1{\cdot}58 \times 4 + 15 \times 1{\cdot}58\}}{1{\cdot}75 \times 5{\cdot}25} - 107 \text{ lbs. sq. in.}$$

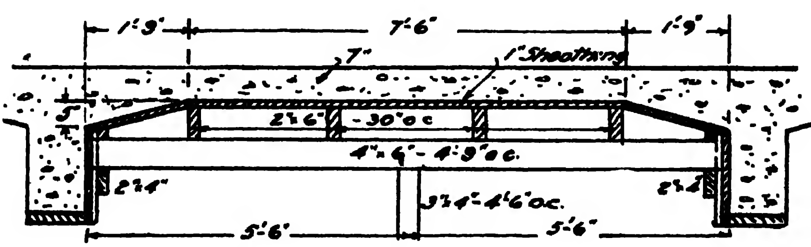
With 3 in. by 6 in.,

$$v = \frac{1{\cdot}5 \times \{100 \times 2{\cdot}5 \times 4 + 15 \times 2{\cdot}5\}}{2{\cdot}75 \times 3{\cdot}25} \quad 174 \text{ lbs. sq. in.}$$

So that either size joist may be used.

Design 9: Floor Slabs with Deep Haunches at the Beams.

Design forms for a 7 in. slab span 11 ft. with a haunch 5 in. deep and 1 ft. 9 in. long at the beams.



DESIGN 9

When the haunch is deep it is often better to run the joists longitudinally and carry them on ledgers supported by the beams and so avoid the waste of cutting.

The haunch sheathing can span from beam side to the first joist at the end of the haunch; the average slab thickness will be 9½ in. and 1 in. sheathing will be sufficient.

Depth of joist should be about the same as that of haunch (not less), and will be continuous.

From Table 4, 2 in. by 6 in. at $26 \times 1{\cdot}25 = 32\frac{1}{2}$ in. will span 4 ft. 9 in.

From Table 1, 1 in. sheathing will span $29\frac{1}{2} + 1\frac{1}{2} = 31$ in., so this governs; however, since total width of slab is 7 ft. 6 in. we will use 3 spaces at 30 in.

Loads on ledger (assuming same for all joists) $= (84 + 40) \times 2{\cdot}5 \times 4{\cdot}75 = 1475$ lbs. The span of 11 ft. will be too great without a centre post. With centre row of posts, from Table 5 a 4 in. by 6 in. will carry the load over a span of 5 ft. 6 in.

Shear at end of ledger (approx.)

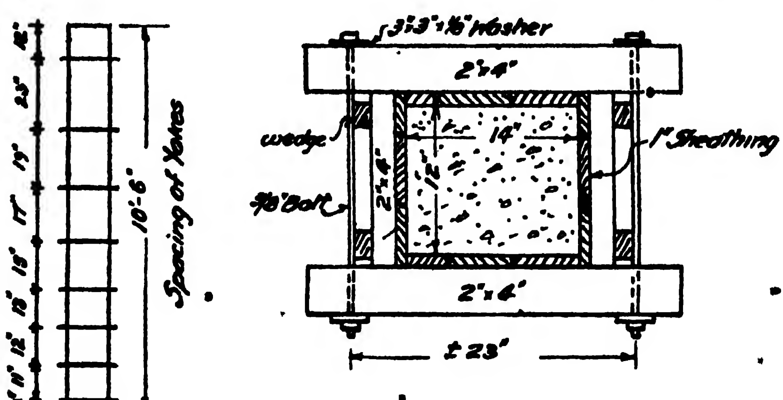
$$= 290 \times 4{\cdot}75 \times \frac{3{\cdot}75}{5{\cdot}5} + 310 \times 4{\cdot}75 \times \frac{1{\cdot}25}{5{\cdot}5} = 1235,$$

width of beam ledger $= \dfrac{1235}{1{\cdot}75 \times 400} = 1{\cdot}76$; use 2 in. by 4 in.

Load on centre posts will be 3160 lbs.; a 3 in. by 4 in. can be used.

Design 10: Small Column Form.

Design form for a column 12 in. by 14 in., 10 ft. 6 in. high.



DESIGN 10.

The form will not be used over twice, so 1 in. sheathing will be used.

From Table 6, a line drawn through depth of 10 ft. 6 in. will give the first spacing for a 14 in. column at 11 in. Since the first yoke should be about 4 in. up from the bottom, the spacing will be 4 in., 11 in., 12 in., 13 in., 15 in., 17 in., 19 in., 23 in. This will give the top yoke 12 in. below the top of the column. Yokes spanning the 12 in. dimension can be 2 in. by 4 in. on the side instead of edge.

To show the size of bolts and washers theoretically required, they will be calculated from formulæ 43, 44 and 45.

Maximum stress in bolts will be in the second yoke up, and diameter of bolt $d = \sqrt{\frac{9{\cdot}25 \times {\cdot}96 \times 1{\cdot}17}{175}} = 0{\cdot}243$ in., say $\frac{1}{4}$ in.

As this is a small column a $\frac{1}{4}$ in. bolt could be used, though a $\frac{5}{8}$ in. bolt would be better, as mentioned before.

For $\frac{1}{4}$ in. bolt, hole in washer will be $\frac{3}{8}$ in. diameter.

$$\text{Area of square washer} = \frac{125 \times 9{\cdot}25 \times {\cdot}96 \times 1{\cdot}17}{800} + 0{\cdot}785 \times 0{\cdot}375^2$$

$$= 1{\cdot}733 \text{ sq. ins., or side} = \text{say } 1\tfrac{1}{2} \text{ in.}$$

$$\text{Thickness of W.I. washer} = \frac{1}{220}\sqrt{\frac{400 \times (2{\cdot}25 - 0{\cdot}44^2) \times 0{\cdot}53}{0{\cdot}44}}$$

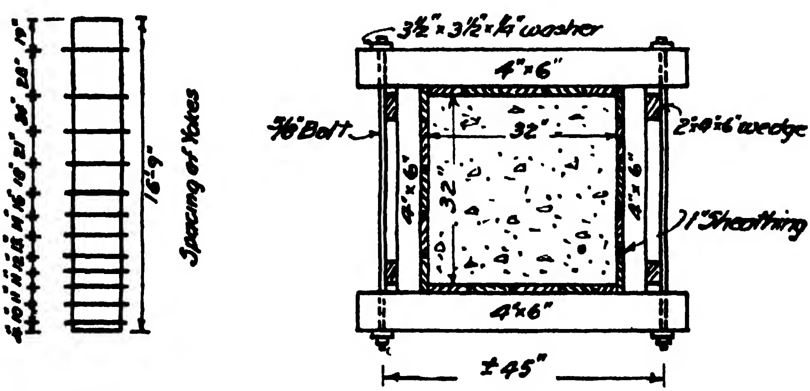
$$= 0{\cdot}143 \text{ in.}$$

$$= \text{say, } 5/32 \text{ in.}$$

It would be better to standardise on a larger size according to the rule given in Chapter III, and use a $\frac{5}{8}$ in. bolt with a 3 in. by 3 in. by $\frac{3}{8}$ in. washer.

Design 11 : Large Column Form.

Design form for a 32 in. square column, 16 ft. 9 in. high.

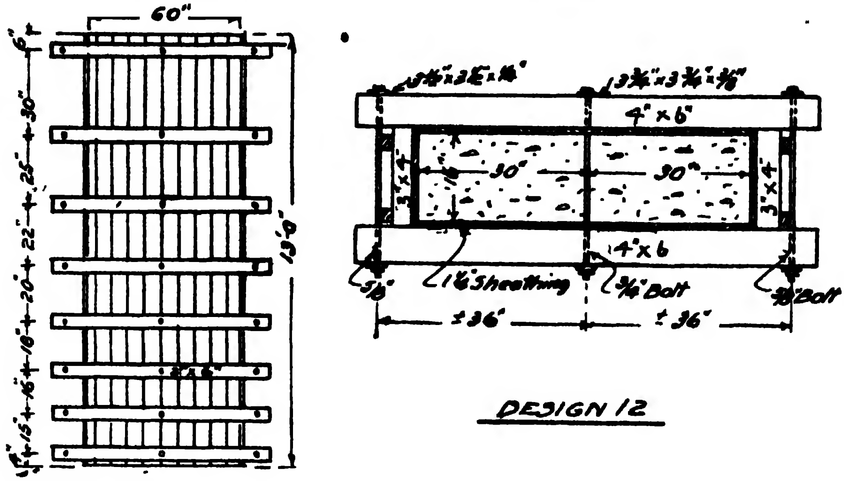


DESIGN 11.

It will be seen from Tables 6 to 9 that a 4 in. by 6 in. yoke should be used. According to the table, the first yoke will be 2 in. above the bottom of the column. Change this to 4 in. and space as given, 10 in., 11 in., 11 in., 12 in., etc., using $1\frac{1}{4}$ in. sheathing. Top yoke will be 19 in. from top of column. The other pair of yokes may be 4 in. by 6 in. on side instead of edge. Bolts will be $\frac{5}{8}$ in. diameter and washers $3\frac{1}{2}$ in. by $3\frac{1}{2}$ in. by $\frac{1}{4}$ in.

Design 12: Very Large Columns.

Design forms for a column 16 in. by 60 in., 13 ft. high.



DESIGN 12

To find what size yoke would be required without a centre support apply formula 37, assuming it is desired not to space the yokes closer than 9 in. If first yoke is 6 in. up, $h = 12\cdot5$, $l = 5 + 1 = 6$ and $s = 9 = \dfrac{17\cdot06 \times bd^2}{12\cdot5(6^2-1)}$, from which $bd^2 = 231$.

If we use a yoke 4 in. wide, depth $= 7\cdot83$, or 4 in. by 8 in. is required. With this method of framing a width less than 4 in. would not be desirable.

For a deflection of ⅛ in., formula 41, $d = \dfrac{32 \times 6^4}{6^2 - 1} = 11\cdot85$, so that deflection governs and a 4 in. by 12 in. must be used. This could be two 2 in. by 12 in., with bolt between. This, however, is too large practically, so a centre bolt will be used, and the spacing can be that for a 30 in. column.

From Table 9, using 4 in. by 6 in. yokes, placing first yoke 4 in. up, spacing will be 15 in., 16 in., 18 in., 20 in., etc., with 1¼ in. sheathing.

Load on centre bolt at second yoke up

$$= 125 \times 11\cdot42 \times 1\cdot29 \times 2\cdot5 \times 2 \times \frac{1\cdot75}{3} = 5360 \text{ lbs.}$$

Diameter of threaded bolt $d = \dfrac{5360}{14000 \times 0\cdot785}$ $0\cdot70$, say, ⅞ in. bolt.

Area of washer $\dfrac{5360}{400} - 0\cdot785 \times 0\cdot875^2 = 14$, or side $= 3\frac{3}{4}$ in.

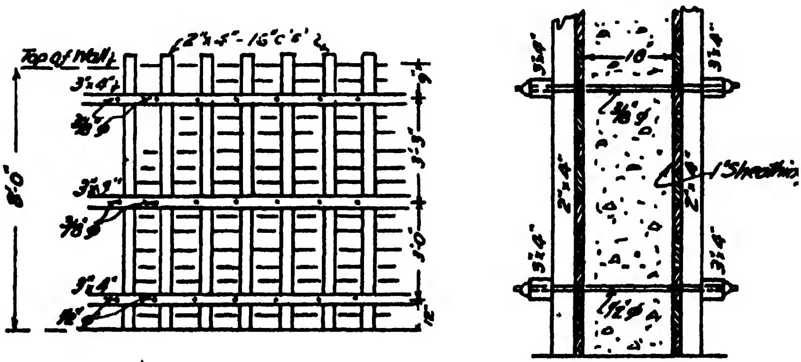
Thickness $= \dfrac{1}{220} \dfrac{400 \times 12\cdot82 \times 1\cdot3125}{1\cdot125} = 0\cdot352$, say, ⅜ in.

According to our rule the standard washer would be 4 in. by 4 in. by ⅜ in. The two end bolts may be ⅝ in. and the washers 3½ in. by 3½ in. by ¼ in.

The ends being only 16 in., smaller yokes may be used. We find from the tables that a 4 in. by 4 in. yoke will give approximately the same spacing for a 16 in. column as we are using for the 60 in. side, and therefore being end yokes we can use a 3 in. by 4 in. on its side.

Design 13: Low Wall Forms.

Design forms for an 18 in. wall, 8 ft. high.



DESIGN 13

From Table 2, 1 in. sheathing will span 14 in., add 2 in. for stud, so place studs 16 in. on centre.

From Table 10, using 2 in. by 4 in. studs, the spacing of the ties or wales will be 12 in., 36 in., and 39 in.

Load on lowest tie $= 7 \times 125 \times 1{\cdot}333 \times 2{\cdot}5 = 2915$ lbs.

Size of plain tie rod $d = \sqrt{\dfrac{2915}{20000 \times 0{\cdot}785}} = 0{\cdot}43$, or, say, ½ in. diameter.

Load on second tie $= 4 \times 125 \times 1{\cdot}333 \times 3{\cdot}125 = 2085$ lbs., requiring a ⅜ in. rod, and the same size would be used for the top wale or tie.

If the rods are to be drawn after stripping it would be better to use ½ in. rods throughout to avoid having more than one size on the job.

Wire is commonly used for tying low wall forms, but the factor of safety is only about half what it is when bolts are used, since the wire is stressed up nearly to the elastic limit and to about one-half of its ultimate strength. Wires will seldom break from the pressure of the concrete, but they are liable to snap from sudden jars, as when hit by large stones or "plums" dropped into the wall, and they will cut into

the stud or wale around which they are twisted, causing the form to give more or less.

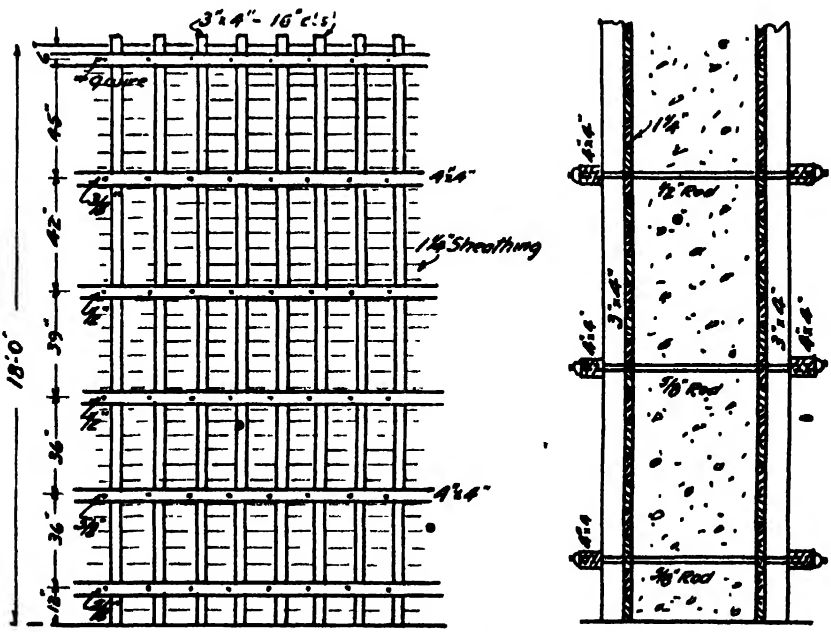
Assuming a stress of 40,000 lbs. per sq. in. in the wire, the area of wire required would be, for the lowest wale = 0·073 sq. in., and the sectional area of No. 9 wire being 0·0163 sq. in., four wires would be required, or two double strands. These strands are twisted tightly together inside the wall.

The same would be required for the centre wale and one double strand for the top wale.

Since with plain round rods it is assumed that patent clamps will be used, it is not necessary to use washers, as the clamp will give sufficient bearing.

Design 14: High Wall Forms.

Design forms for a wall 18 ft. high, concreting to be completed in one operation.



DESIGN 14

From Table 2, 1¼ in. sheathing will span 13½ in., using 3 in. by 4 in. studs; and, adding the thickness of the stud to the span of the sheathing, the studs will be 16 in. centre to centre.

From Table 10, spacing of the wales from the bottom will be 12 in., 36 in., 36 in., 39 in., 42 in. and 45 in., the last wale being 6 in. from the top of the wall.

			lbs.
Load from each stud on 1st	wale	$= 17 \times 100 \times 1{\cdot}33 \times 2{\cdot}5$	$= 5660$
" " " 2nd	"	$= 14 \times 100 \times 1{\cdot}33 \times 3$	$= 5600$
" " " 3rd	"	$= 11 \times 100 \times 1{\cdot}33 \times 3{\cdot}125$	$= 4600$
" " " 4th	"	$= 7{\cdot}75 \times 125 \times 1{\cdot}33 \times 3{\cdot}375$	$= 4400$
" " " 5th	"	$= 4{\cdot}25 \times 125 \times 1{\cdot}33 \times 3{\cdot}625$	$= 2600$

The exact loads by formulæ 52 and 53 will be successively 5530, 5600, 4545, 4305, 2515, and for the top wale 565 lbs. It will be noticed that these are very close to the loads found by the approximate method.

To find what size wale will be required if only alternate studs are bolted, $M = \frac{5600 \times 2{\cdot}66 \times 12 \times 8}{4 \times 10} = M_r = \frac{1600 \times bd^2}{6}$, or $bd^2 = 134$, or a 4 in. by 6 in. is required.

Load on the tie $= 5600 \times 2 = 11200$ lbs., and for threaded bolts

$$d = \sqrt{\frac{11200}{14000 \times {\cdot}785}} = 1 \text{ in. diameter, or with plain rods}$$

$$d = \sqrt{\frac{11200}{20000 \times {\cdot}785}} = 0{\cdot}84 \text{ in., or, say, } \tfrac{7}{8} \text{ in. round rod.}$$

With 1 in. bolt, area of washer $= \frac{11200}{400} + 0{\cdot}785 \times 1{\cdot}125^2 = 29$ sq. in., or, say, $5\frac{1}{2}$ in. square.

The thickness of washer $t = \frac{1}{220}\sqrt{\frac{400(30{\cdot}25 - 1{\cdot}5^2) \times 2}{1{\cdot}5}} = 0{\cdot}55$, say, $\frac{1}{2}$ in. thick.

The wale, bolt and washer have been calculated to show the large sizes required. It would be better to use smaller sizes and tie each instead of alternate studs.

The wales can now be made 4 in. by 4 in., as they will only act as stiffeners.

The load now on the tie will be 5600 lbs., and for plain rods

$$d = \sqrt{\frac{5600}{20000 \times 0{\cdot}785}} = 0{\cdot}60 \text{ in., or, say, } \tfrac{5}{8} \text{ in. diameter.}$$

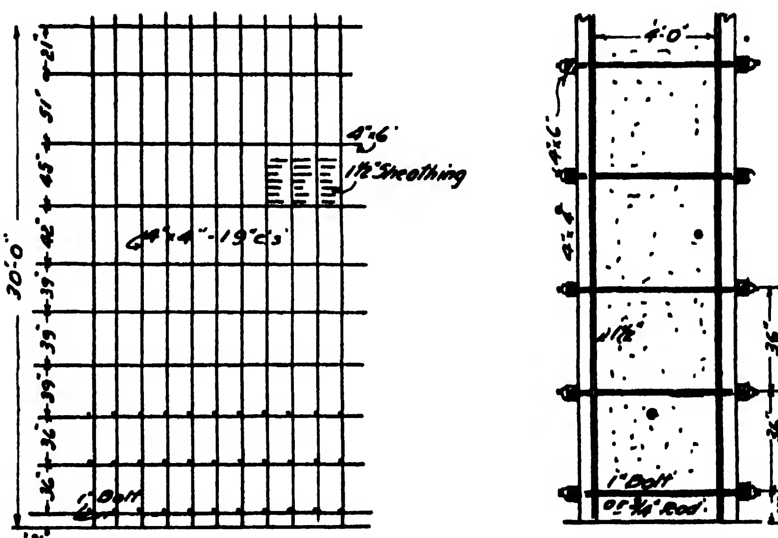
The two lower wales will need $\frac{5}{8}$ in. rods, the third and fourth $\frac{1}{2}$ in. rods, and the fifth $\frac{3}{8}$ in. rods, or two double strands of No. 9 wire and the top one double strand of wire.

Design 15: Very High Walls and Piers poured Monolithic.

Design forms for a pier 4 ft. thick and 30 ft. high, no construction joint being allowed.

We will use $1\frac{1}{2}$ in. sheathing, and spacing from Table 2 is 15 in. Using 4 in. by 4 in. studs, they can therefore be placed 19 in. on centre.

From formula 50, if first wale is placed 12 in. up, spacing of next wale, $s = 66 \sqrt{\dfrac{3{\cdot}75 \times 3{\cdot}75^3}{75 \times 29 \times 1{\cdot}58}}$ 32·2 in.; allowing for the wale and for the fact that we have assumed uniform pressure between wales, say, 36 in. Spacing of third wale, $s = 66 \sqrt{\dfrac{3{\cdot}75 \times 3{\cdot}75^3}{75 \times 26 \times 1{\cdot}58}} = 33{\cdot}2$, say, 36 in. Similarly the spacing of the remaining wales will be 39 in., 39 in., 39 in., 42 in., 45 in. and 51 in.



DESIGN 15

		lbs.
Loads brought by studs to lowest wale	$= 29 \times 75 \times 1{\cdot}58 \times 2{\cdot}5$	$= 8600$
,, ,, ,, ,, second ,,	$= 26 \times 75 \times 1{\cdot}58 \times 3$	$= 9200$

Diameter of threaded bolt $= \sqrt{\dfrac{9200}{14000 \times {\cdot}785}} = 0{\cdot}91$, say, 1 in. diameter, and for plain rod $= 0{\cdot}76$ in., or, say, ¾ in. diameter.

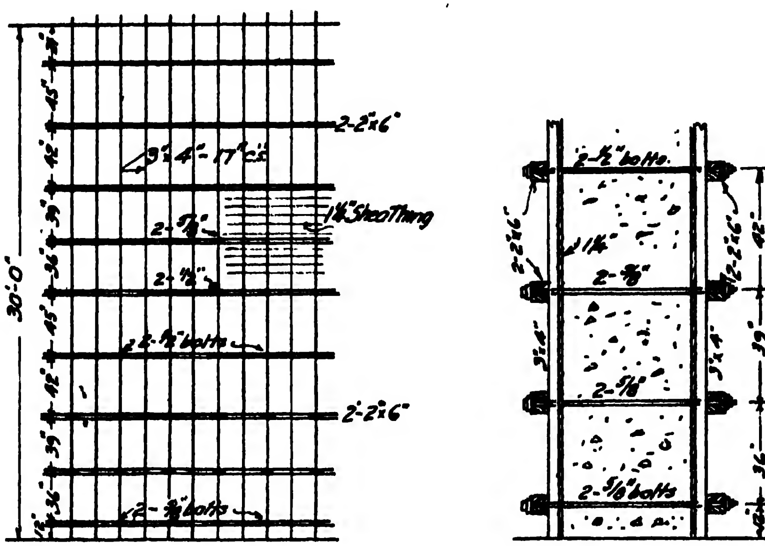
The size of rods or bolts for the other wales are similarly calculated. The wales should be 4 in. by 4 in. at least, or preferably 4 in. by 6 in.

With high walls or piers poured monolithic ties will be required on every stud or very large bolts and washers, which are not economical, will be necessary; wires should not be used.

Sometimes a large bolt is replaced by two smaller bolts, placed one each side of the stud (*see* Design 16).

Design 16 : Very High Walls or Piers poured in two Operations.

Design forms for a pier 4 ft. wide and 30 ft. high, the concrete to be poured in two operations, but the forms to be built up the full height.



DESIGN 16

Using 1½ in. sheathing, for 15 ft. high the spacing of the studs will be 14½ + 2½ = 17 in. Using 3 in. by 4 in. studs, wales will be spaced at 12 in., 36 in., 39 in., 42 in., 45 in., 36 in., 39 in., 42 in. and 45 in., the top wale being 24 in. from the top of the wall.

		lbs. from each stud.
Load on lowest wale	= 14 × 100 × 1·42 × 2·5	= 4960
„ 2nd „	= 11 × 100 × 1·42 × 3·125	= 4890
„ 3rd „	= 7·75 × 125 × 1·42 × 3·375	= 4640
„ 4th „	= 4·25 × 125 × 1·42 × 3·625	= 2740

With ties at alternate studs, load on tie = 9920 lbs., and calculating the size of wale required, as in Design 14, a 4 in. by 6 in. will be required. Instead of using one large tie we will use two small ones, and the load on each will be 4960 lbs.; the diameter of plain rod required is 0·55, or, say, ½ in., or if threaded bolts are used the diameter will be ⅝ in. One rod or bolt will be placed each side of the stud and between the two 2 in. by 6 in., forming the 4 in. by 6 in. wale.

Use the same sizes for the three lower wales. For the 4th wale two ½ in. bolts would be sufficient, but it would be better to use the same sizes as before.

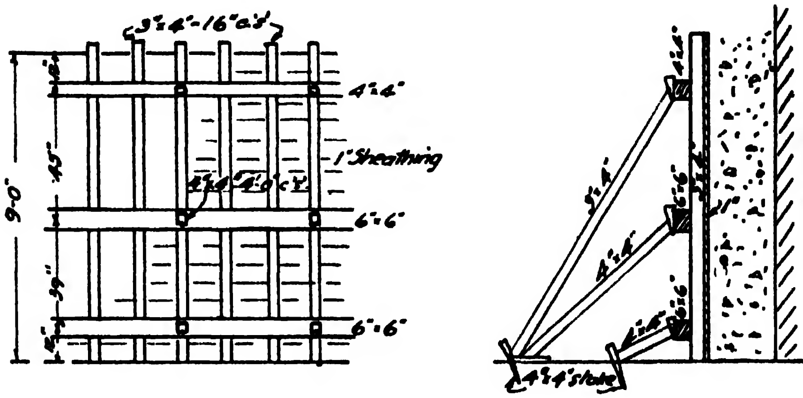
The loads on the 5th wale will be greater from the upper pour and

(approx.) $= 13{\cdot}75 \times 100 \times 1{\cdot}42 \times 2{\cdot}5 \times 1{\cdot}25/3 = 2040$ lbs. (exactly 2090). Use $\frac{1}{2}$ in. or $\frac{5}{8}$ in. bolts.

The bolts in the wales for the upper half of the wall will be similar to those on the lower half.

Design 17: Wall Forms without Internal Ties.

Design forms for a wall 9 ft. high poured against an existing wall, so that a form for only one side will be required.



DESIGN 17

When the mass of concrete to be poured is very large, as in bridge abutments, or when a new wall is to be poured against an old one so that only one side can be formed, it is impossible to use internal ties, and the forms have to be held by external braces.

With 1 in. sheathing, 3 in. by 4 in. studs will be 16 in. centre to centre, and wales will be placed 12 in., 39 in. and 45 in. up.

	lbs.
Load on each stud at lowest wale $= 8 \times 125 \times 1{\cdot}33 \times 2{\cdot}625$	$= 3500$
" " " " 2nd " $= 4{\cdot}75 \times 125 \times 1{\cdot}33 \times 3{\cdot}5$	$= 2775$
" " " " 3rd " $= 2{\cdot}875 \times 125 \times 3{\cdot}75 \times \dfrac{1{\cdot}47}{3{\cdot}75} \times 1{\cdot}33 + \dfrac{125}{2} \times 1 \times 1{\cdot}33$	$= 785$

The centre of pressure between the 2nd and 1st wales from the top is 1·47 ft. up from the 2nd wale.

Bracing the lowest wale at every third stud or at spaces of 4 ft., to find size of wale required $M = 3500 \times 1{\cdot}33 \times {\cdot}8 \times 12 = M_r = \dfrac{1600bd^2}{6}$.

or $bd^2 = 168$, or size can be 6 in. by 6 in.

The same size will be used for the middle wale with the braces the same distance apart.

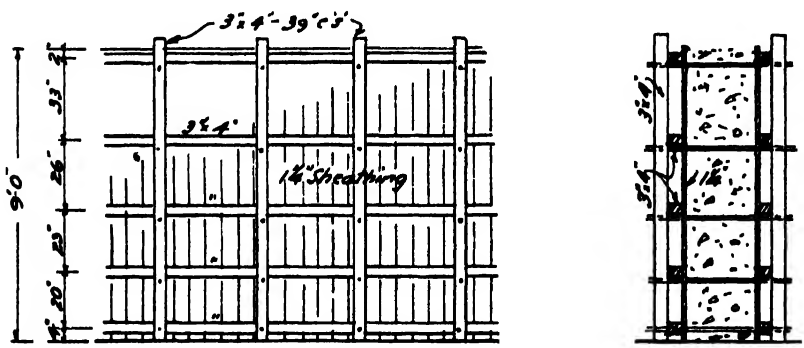
For the upper wale for the same span of 4 ft. a 4 in. by 4 in. wale will be sufficient.

The angle of the brace to the lower wale will be about 30 degrees, and thrust = 3500 × 3 × 1·25 = 13125 lbs. The centre brace will be placed at 45 degrees, and thrust = 2775 × 3 × 1·5 = 12500 lbs. The top brace will be at 60 degrees, and thrust = 785 × 3 × 2 = 4710 lbs. Length of braces will be 2 ft., 4 ft., and 9 ft. respectively.

From Table 11, assuming hardwood wedges are used between brace and wale, 4 in. by 4 in. braces can be used for the bottom and centre wale, and 3 in. by 4 in. braces for the top wale.

Design 18: Wall Forms with Horizontal Studs.

Design forms for a wall 9 ft. high using 1¼ in. sheathing placed vertically.



DESIGN 18

Placing first horizontal stud 4 in up, from Table 2 or 3 the next spacing will be 17 in. + 3 in. for stud, or 20 in. total; next spacing will be 19½ + say 3½ = 23 in.; next spacing will be, from formula 46, for a depth of 9 ft. − (3 ft. 11 in.) = 5 ft. 1 in.,

$$l = \sqrt{\frac{166{\cdot}67 \times 12 \times 1{\cdot}0625^2}{125 \times 5{\cdot}08}} = 22\tfrac{1}{2} + 3\tfrac{1}{2} = 26 \text{ in.}$$

Similarly, next spacing will be 33 in.

Since we assume that the pressure is constant between any two studs while it decreases rapidly when the spacing is large, it is quite safe to add at least 3 in. to the calculated spacing.

		lbs.
The load on the bottom stud (approx.)	$= 8{\cdot}67 \times 125 \times 1{\cdot}17 =$	1270
,, ,, ,, 2nd ,, ,,	$= 7 \times 125 \times 1{\cdot}79 =$	1570
,, ,, ,, 3rd ,, ,,	$= 5{\cdot}08 \times 125 \times 2{\cdot}04 =$	1295

The loads on the two upper studs will be less, so the second stud will govern the spacing of the vertical wales or of the ties.

Using 3 in. by 4 in. studs, to find maximum span for a deflection of $\frac{1}{8}$ in., by formula 14, $\frac{1}{8} = \frac{3 \times 1570 \times l^4 \times 12^4 \times 12}{384 \times 12 \times 1{,}200{,}000 \times 2{\cdot}75 \times 3{\cdot}75^3}$, or $l = 3{\cdot}03$ ft., say, 3 ft. without wales and 3 ft. 3 in. with wales.

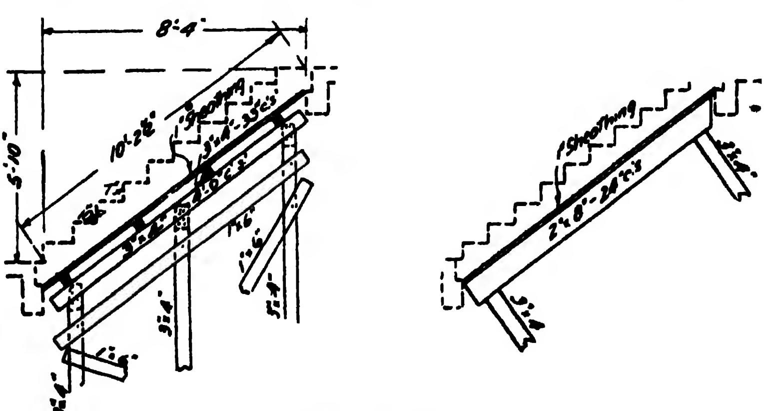
If no wales are used, or if wales are only used to keep the form in line, all the horizontal studs will be tied at this spacing.

If it is desired to take advantage of the smaller loads on the upper studs, short lengths of wale can be used, one from the lowest to the third stud at above spacing and another from the third to the top stud at a greater spacing depending upon the maximum span for the fourth stud. Load on this stud $= 2{\cdot}92 \times 125 \times 2{\cdot}46 = 900$ lbs. per lin. ft., and calculated as above $l = 3{\cdot}5$ ft., or, with wales, say, 3 ft. 9 in. As this is not much greater than for the lower wales it would be better to run the wales through to the top at the smaller spacing.

The size of ties is calculated as in previous designs.

Design 19: Stair Forms.

Design forms for a flight of stairs 5 ft. 10 in. high from landing to landing 4 ft. wide with 10 in. treads and 7 in. risers, the slab being 5 in. thick.



DESIGN 19

This design is typical for inclined slabs.

Weight of slab 60 lbs., steps 35 lbs., live load, say, 40 lbs., or total of 135 lbs. sq. ft. Not more than 40 lbs. need be taken for live load because there will be no wheeling over the slab.

The dead load will act vertically, while the sheathing and joists will be inclined so that they will not receive the full load, and the problem is to find the load for which they should be designed; then the remaining calculations are as before.

When the spans are measured along the incline, the vertical load can be reduced by the ratio of the horizontal run to the inclined span of the stairs (which can be scaled or $= \sqrt{8{\cdot}33^2 + 5{\cdot}83^2}$), or in this case by $\frac{8{\cdot}33}{10{\cdot}2} = 0{\cdot}816$.

Load to be used for designing is then $135 \times 0{\cdot}816 = 110$ lbs. sq. ft.

To make use of Table 1, deduct 75 lbs. for live load leaving 35 lbs. or the equivalent of a 3 in. slab, and span for 1 in. sheathing is $32\frac{1}{2}$ ins., say, 33 in.

The span of the joists will be 4 ft., and using 3 in. by 4 in. from formula 5, $s = \frac{1600 \times 2{\cdot}75 \times 3{\cdot}75^2}{110 \times 16} = 35$ in. Therefore strength of sheathing governs spacing, which will be 33 in.

Load carried to each ledger $= 110 \times 2{\cdot}75 \times 2 = 605$ lbs.

From Table 5, for 605 lbs. at 33 in. centre to centre a 2 in. by 6 in. will span 6 ft., which will be the spacing of the posts. However, with one post at the centre the span will only be 5 ft., and 3 in. by 4 in. ledgers can be used.

Posts should be at right angles to the slab, or if they are placed vertical they should be well braced and prevented from kicking over at the top.

If it is desired to run the joists longitudinally without using ledgers the span will be 10 ft. and for $s = 24$ in., $24 = \frac{1600 \times bd^2}{110 \times 100}$ and $bd^2 = 163$, or 2 in. by 8 in. is required.

Posts can be 3 in. by 4 in.

CHAPTER VI.

DETAIL CONSTRUCTION OF FOOTING FORMS.

In previous chapters we have considered the questions of loads and stresses and the method of calculating and choosing the correct timber sizes and spacing in order safely to carry the loads.

We now come to the practical details of assembling and erecting the forms for the various members that constitute a reinforced concrete structure. The methods that will be described are, of course, not the only ones, but they are the results of years of development and have become recognised as standards and are used by the majority of large concrete contracting firms.

The two main thoughts governing the methods of construction are (1) ease of stripping, and (2) unit construction.

The method of stripping, although it is the last operation, is the first consideration, since that is where time and money can be saved in both labour and material.

By unit construction is meant the building up of complete units, such as beam and column sides, wall and slab panels, etc., at the bench away from the points of erection, ready for erection as required. By this means most of the carpenter work is done under the best conditions with the aid of a power saw, leaving little to be done on the job where working conditions are not so good except the fitting together of the various units. Also, the building of the units can be started in advance of the date they will be required, so that there will be no delay in operations. Of course on a small job, where the forms will only be used once, it may often be more economical to build and erect the forms at the same time at the place where they are required.

Estimating Cost.—This subject would fill a book itself, and only sufficient information can be given to form a general guide and to illustrate a satisfactory method of estimating. Labour cost varies greatly, but for an average size job, using the methods of construction given here and with good organisation, the figures should not vary much from those given. Small jobs will cost more and large jobs less, as they can be better organised.

Each contractor should keep his own costs on performing different operations. This is easily done by having the timekeeper, or preferably the carpenter foreman, report at the end of each day the number of hours spent on all the operations, and at the end of each week or month the work done is measured up and the cost calculated.

It is convenient to have a unit of measure for timber regardless of its size, and to apply an average cost per unit. This unit will be the cubic foot, and the amount of timber required to build any forms will be expressed in the number of cubic feet required per square foot of *contact with the concrete*, allowing for matching, waste, cleats, etc. Note it is the contact area that is measured and not that of the forms, since the first is definite and can be taken from the plans.

The amount of timber required per square foot does not vary much with the size of a member, so that the average value can be applied to all similar members regardless of their size. This amount will consist of about 1 part of sheathing to 1 to 1½ parts of timber, which determines the average cost per unit.

Labour cost is best calculated from the number of man-hours required to build the various kinds of forms and applying the current labour scale for carpenters and labourers. The number of man-hours must be divided between carpenters and labourers, and will vary in different parts of the country according to the rules of the trade unions. It will be assumed that the labourers will do all carrying and handling of timber, help in erection, and do most of the stripping. Timber piles will be assumed to be within 200 ft. of the place of assembly.

Wall Footing Forms (*Fig.* 8).—These are the simplest of all forms to build, being merely a heavy plank or light built-up panel set to line and staked in position.

Old timber, if available, is usually used, old 12 in. plank (or less) is very suitable for depths less than 12 in. One side should be set to line and held with stakes, about 6 ft. apart, the other side being set from this by 1 in. by 4 in. spreaders, cut to the width of the footing and knocked out as concreting reaches them. New planks should not be used for this purpose.

For deeper footings up to 2 ft. 6 in., which is about the maximum, the sides can be made of 1-in. timber built into panels about 8 ft. long, with battens 2 ft. on centres to hold the boards together. For heights up to 18 in. use 1 in. by 4 in. battens nailed on flat; for heights from 18 in. to 30 in. use 2 in. by 4 in. battens nailed on flat. The panels can then be held to line by 2 in. by 4 in. wales top and bottom, which can be braced back to stakes or the sides of the trench about every 6 ft.

The side panels need not be the exact depth of the footing, but should be as much greater as is necessary without sawing the boards. To space and line one side from the other 1 in. by 4 in. spreaders are used. Sometimes the trench is backfilled against the forms, which saves the bracing although it increases the cost of stripping. The forms can be stripped after a few hours and moved forward.

Column Footing Forms.—There are three standard types of column or pier footings: the square or rectangular footing, the stepped footing, and the sloping side footing, the two latter being variations of the former in order to save concrete. They are all built in a similar manner of 1 in. tongued and grooved sheathing cleated together, the four sides being

built as units, assembled at the place of erection, and fastened together. Cleats or battens should be 24 in. apart, and the size depends on the depth of the footing, using 1 in. by 4 in. up to 18 in. deep; 2 in. by 4 in. for depths from 18 in. to 30 in.; and 2 in. by 4 in. on edge for depths from 30 in. to 48 in.

Square or Rectangular Footings (*Fig.* 9)—There are two ends

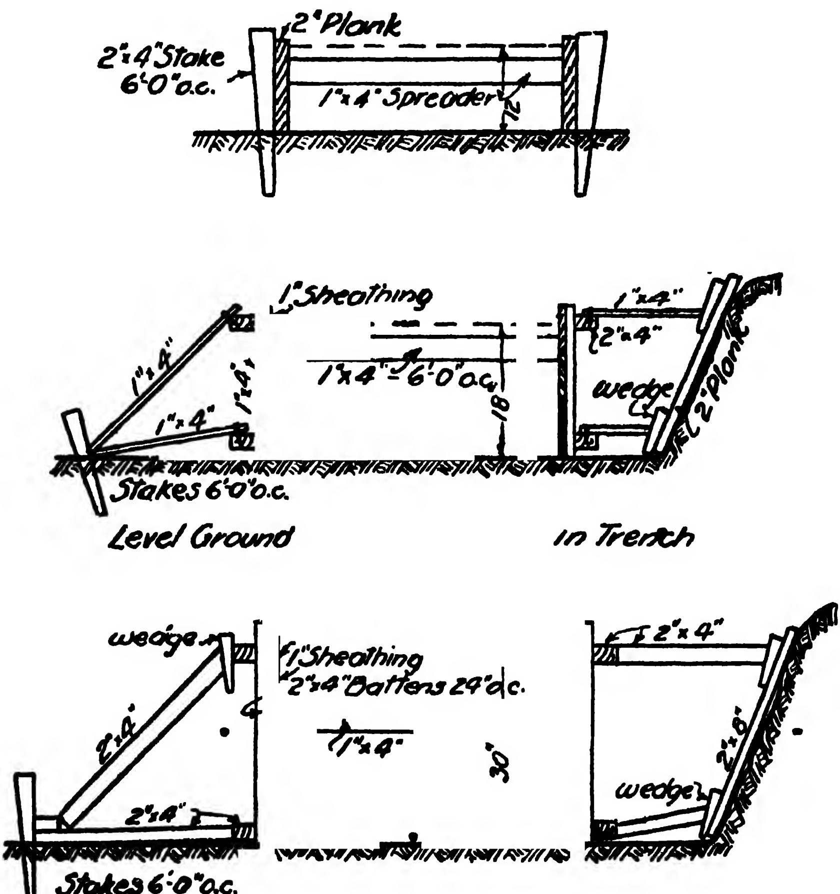


Fig. 8. Wall Footings

and two sides, and the ends are the short sides of rectangular footings. The ends are built to the exact dimension (A) of the footing, with a cleat each end on the outside placed a distance from the ends of the sheathing equal to the depth of the cleat used with a slight allowance for clearance; that is, if the cleats are 2 in. by 4 in. dressed the cleats on the ends will be 2 in. from the ends of the sheathing. The two sides are built about 12 in.

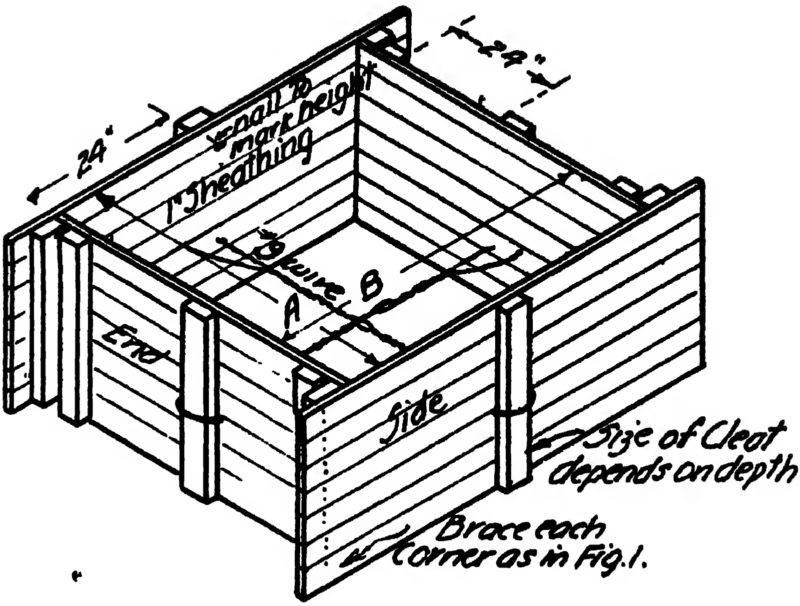


Fig. 9. Square Footing

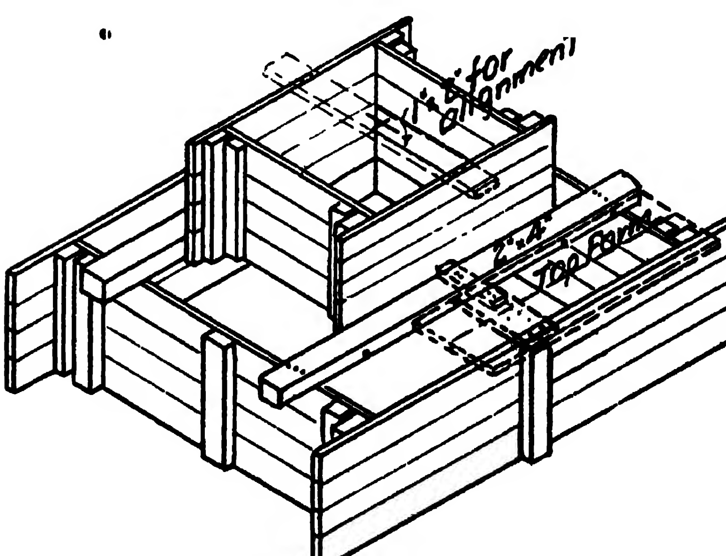


Fig. 10. Stepped Footing

longer than the dimension of the footing, although all the boards need not necessarily be sawn to the same length, and are cleated each end on the inside, the distance between cleats being the dimension (B) + twice the thickness of the sheathing, using the dressed thickness.

One or two intermediate outside cleats will be necessary on sides and ends, so that the distance between cleats is not over 24 in. The size of the cleats will depend on the depth of the footing, as above.

The side and end panels should be made up on the bench, with holes bored for the wires if they are used, all ready to be carried to the place of erection when required.

The four panels are assembled by butting the end panels against the end cleats on the sides. If they are to be used only once or twice the panels can be nailed together through the cleats; if this method is followed the end cleats on the end panels are sometimes omitted, except one cleat at the corner which is to be opened for stripping. This corner is nailed together from the outside with long spikes through both cleats, and to strip only this corner is opened sufficiently far to enable the form to be removed as a whole.

This cannot be done successfully with very large forms or when the forms are to be used several times over, in which cases the panels should not be nailed together. The corners should be braced back to stakes or to the side of the excavation as in *Fig.* 8, bracing only being required on the sides. The intermediate bracing can be done in the same way, or opposite cleats can be wired together as in *Fig.* 9. The wires should be placed about a third of the way up and should consist of a double strand of No. 9 or 10 wire, and if the footing is over 3 ft. deep two double strands should be used. The corners can also be held by wiring together the end cleats on the sides, and no exterior bracing need be used. When wires are used the reinforcing steel must be placed before the wires.

No labour should be spent in cutting the panels to the exact depth; full width boards should be used, and a nail driven in the side to show the depth of the footing.

Stepped Footings (*Fig.* 10).—To save concrete a deep footing is often built in two or three steps of decreasing size. Each step will be similar to the form for a square or rectangular footing described above, the only point to note being the method of placing and supporting the successive forms.

The largest and lowest form is placed first, and usually filled and the concrete allowed to set a little before placing the next form. The upper form can then be held by placing a 2 in. by 4 in. on each side, nailed on to the top of the sides of the lower form.

If it is desired to pour all the steps together, the space between the different forms must be sheathed over with a top form to prevent the concrete from overflowing. This top form is shown in dotted lines in *Fig.* 10; it is built out of two boards cleated together and held down by cleats to the 2 in. by 4 in. As there will be considerable upward pressure on the top form it should be well weighted down.

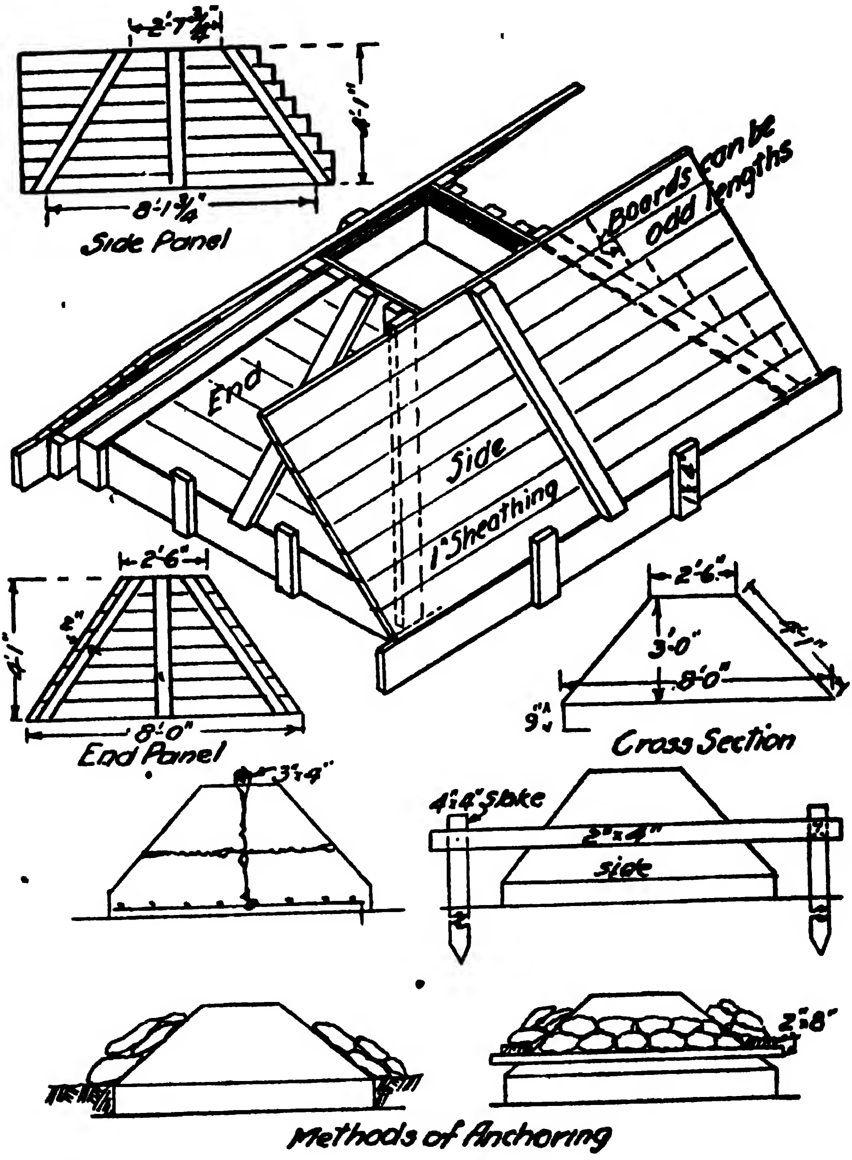


Fig. 11. Sloping Side Footing.

Sloping Side Footings (*Fig.* 11).—Footings with four sloping sides are built in the same manner as when the sides are straight. The two ends are cut to the exact dimensions of the footing, first laying out boards of approximately equal length, then marking out the slope lines, putting on the cleats, and sawing through on these lines. The two sides should not be cut to the slope, as this is unnecessary, but the boards can be left of random lengths. Two inside end cleats are nailed on parallel to the slope, allowing a clearance each end from the exact dimension of the side equal to the thickness of the sheathing.

The sloping box is always set on a shallow straight box of the same bottom dimensions The straight box is set first on the ground, securely braced, often by backfilling around it, by wires or by braces to stakes. The reinforcing steel is placed and the sloping form set on top of the box. Its alignment will be fixed by that of the straight form, the cleats on which should extend a little above the sheathing to hold the sloping form.

It is very important that the form be prevented from lifting due to the pressure of concrete on the sloping sides; considerable trouble will ensue if this is neglected To prevent this uplift a wire or wires can be embedded in the centre of the lower portion, fastened to a spike in the piles if piles are used, or to the reinforcing steel if heavy enough to resist the uplift, and twisting the loose ends around a 3 in. by 4 in. across the top of the form. A few bags of sand or cement or heavy stones placed on the sides will answer the same purpose, or if the ground is firm and dry a 2 in by 4 in. can be nailed halfway up the form on each side, nailing the ends to stakes in the ground—stakes will not hold in soft ground.

It will be necessary to calculate the height to build the sides and ends, as only the top and bottom dimensions and the height of the footing will be given. It can easily be found by laying out two opposite cross sections of the footing and measuring the length of the sloping sides, these lengths will be the heights to build the panels. The sides should not be built the exact heights, but whatever the boards give without sawing, and the height of the concrete marked by a nail.

Alignment of Footing Forms.—A mason's line or fine wire should be stretched taut along the centre line of the footings, attached to stakes preferably outside the building. The spacing of the footings can then be measured off, tying on a piece of line at each centre, or preferably, if there are a large number of footings, by stretching a similar line through at right angles. Across the top of the form nail a 1 in. by 2 in., mark off the centre line and the centre on it, and adjust the form until these marks coincide with the lines above.

Cost of Footings.—The amount of timber required is the first consideration. There are usually several footings alike, and in an ordinary building only a few different sizes, so that it is only necessary to build forms for a portion of the footings and reuse them, the number

depending on the speed required and the number of different sizes. Footings can be stripped the day after they are poured.

When there are several similar-shape footings differing only in size, if the forms are made up for the largest, they can easily be reduced for the smaller sizes by sawing a strip off each end panel and putting new stop cleats on the side panels. Most forms can be used three or four times over without any loss in time.

Allow $\frac{1}{6}$th cubic foot of timber per square foot of contact area of all the footings and divide this amount by three or four according to the number of times they are to be used over, or if the sizes differ considerably base this amount on so many of the largest sizes.

To make up panels, erect, and strip 100 sq. ft. of forms will require: For making up, 3 hours carpenter time and $1\frac{1}{2}$ hours labourer time; for carrying to place and erection, 2 hours carpenter time and 2 hours labourer time; for stripping, 3 hours for either carpenter or labourer, usually labourers.

Cost per 100 sq. ft. if used once = 5 hrs. carpenter at..........
$6\frac{1}{2}$ hrs. labourer at..........

Timber (used once $= \frac{100}{6}$, say, 17 cu. ft. at........

The labour cost is based on the total area in contact.

If the forms are used more than once the cost of making up the panels is divided by the number of times used and added to the cost of stripping and erecting.

Cost per 100 sq. ft. if used 4 times = $2\frac{3}{4}$ hrs carpenter at......
$5\frac{3}{8}$ hrs. labourer at.......

Timber used 4 times $= \frac{100}{24}$, say, $4\frac{1}{4}$ cu. ft. at....

Note for American Readers.—Whenever under the heading of "Cost" the quantity of timber required is given in "cubic feet," this can be converted into equivalent board measure by multiplying the quantity by 12—thus, 17 cu. ft. = 204 board feet.

CHAPTER VII.

DETAIL CONSTRUCTION OF COLUMN FORMS.

THERE are five usual column shapes, namely, square, rectangular, L-shape, octagonal, and round.

Rectangular columns are nearly always exterior columns and L-shape columns usually exterior corner columns, while the other shapes are used for interior columns.

Except for round columns the general method of framing will be the same for all shapes. The general design of the forms has been shown in previous chapters; the method is the commonest and the most satisfactory and is now considered to be the standard. It is strong, simple to build and strip, easy to reduce, and does not necessarily require any patent device.

The size of timber to use will vary with the height and size of the column and can be chosen from the tables. If a form is only to be used once or twice 1-in. sheathing will be satisfactory, but for use several times $1\frac{1}{4}$ in. or $1\frac{1}{2}$ in. is better as there will be less breakage and waste. The thicker sheathing, too, will give better lines and surfaces. Since poor lines, fins, bulges, etc., are more noticeable on a column than on any other part of a structure, good timber should be chosen, spruce being very suitable, and generally the thicker the sheathing the better the quality. This is particularly true of exterior columns, perhaps the only exterior concrete that will show, and since they have to be well finished it is economical to frame carefully and choose good timber.

The construction of the column head, or the top part into which the floor frames, will depend upon the kind of floor system used. In a multiple-story building it is usual to build only sufficient forms for all the columns on the first floor and use these over again on the upper floors.

There is no part of the formwork for a building where more money can be saved or lost than in the column forms.

Square Columns (*Fig.* 12).—The method of construction will be the same for rectangular, L-shape, and octagonal columns, with slight variations to suit the particular conditions.

A column form consists of two ends and two sides, each built as a unit or panel, the height of the panel being the story height less the slab thickness less the floor sheathing; this height will include the "head," which may or may not be an integral part of the column form. To determine the spacing of the yokes the height need only be taken to the bottom of the deepest beam framing into the column, since concreting

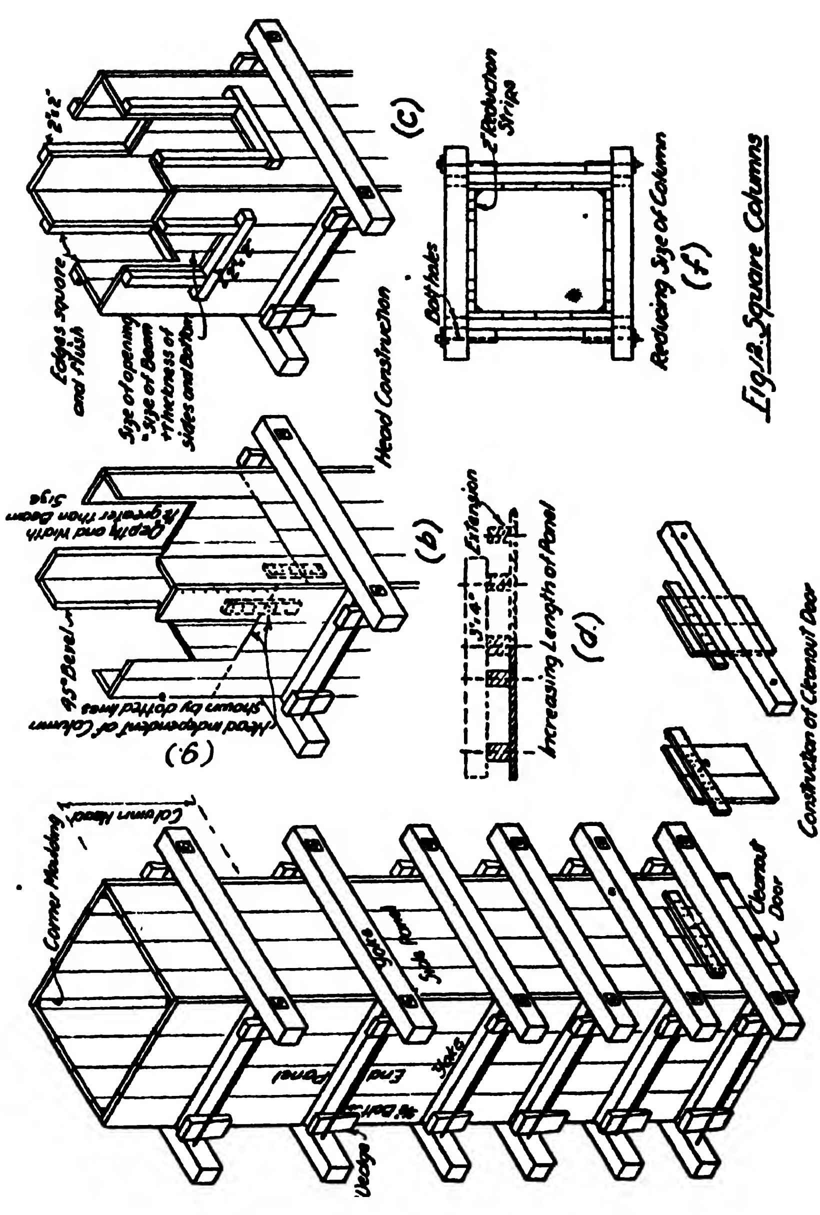


Fig 12. Square Columns

should stop there and the concrete allowed to settle before pouring the beams and slab so that there will be no shrinkage cracks between the top of the column and bottom of the beams.

To facilitate making all measurements and to avoid mistakes in measuring, a " measuring stick " is very useful. This stick is cut the full height from floor to floor, and on it successively from the top are marked out the slab thickness, floor sheathing, beam bottom, girder bottom, yoke spacing, and a mark 12 in up from the bottom. This stick is marked out by the foreman and is used both in assembling the panels and in erection ; it ensures uniformity in measurements, saves the carpenter's time in using a rule, and also prevents his mistakes.

The two ends are built in width equal to the dimension of the column plus twice the thickness of the sheathing, the ends of the yokes and sheathing being flush.

The yokes are first cut to length and laid out on a table at the desired spacing (with the stick) ; then the end board is nailed on at the required distance from the end of the yokes and the remaining boards, except the last one, are clamped tight against the first and nailed ; measurement is then made to determine the width of the last board which is then sawn and nailed on. To obtain a good finished surface it is essential that the boards be drawn up tight before nailing.

The two side panels are similarly made up, but the width of the sheathing will be the same as the dimension of the column, and the yokes will project 8 in. to 10 in. beyond the sheathing at each end The panels should be made up an inch or two longer at the bottom to allow for adjustment due to unevenness and variations in floor level.

The spacing of the yokes must be alike on both ends and sides. Bolt holes are bored in the side yokes about $\frac{1}{8}$ in. larger than the size of bolt, and the spacing is calculated from the size of column and timbers used, allowing about $1\frac{1}{2}$ in. for the wedges. If there is a boring attachment on the machine saw the holes should be bored before the sheathing is attached to the yokes. On a large job each operation should be the work of one man or group of men—sawing and boring the yokes, sawing the sheathing, nailing to the yokes—thereby saving much waste labour, all carrying of timber to the carpenters being done by labourers.

The panels are stacked up ready to be carried into place as required, and they should be oiled at this time so they will not swell and warp if left long in the rain and sun before being used.

Column Heads.—In mushroom slab construction there are no beams and the column will finish off with a splay cap independent of the column, so that the height of the column will be measured to the bottom of the cap and the head of the column will be cut off square, all panels being of the same length (*Fig.* 12 (*a*)).

In other types of construction there will be beams framing into the head, and the head can either be formed as a separate unit or as part of the column form. In the former method (*Fig.* 12 (*g*)) all the panels are stopped off at the bottom of the beams framing into them—these may be

of different heights—and framing the head of the column from the bottom of the beams to the underside of the slab separately from the column. The length of each panel will be measured to the underside of the beam, allowing or not for the thickness of the beam bottom, as will be seen later.

There are several advantages in this method, such as ease of stripping since the column head is not part of the column form ; reduction of panel width on upper floors need only be made on one side of the panel instead of both ; and there is no risk of the openings for the beams being cut to the wrong size, since the head will be formed after the beam bottom is in place.

The head is formed by building four corners of sheathing, nailing each corner together, and cleating them on to the column sheathing. They will be held on the sides by the beam sides, and at the top by the floor sheathing, and as an additional brace a 1 in. by 6 in. is nailed diagonally across the corner from beam side to beam side. In stripping, the cleats are knocked off and the head remains in place while the panels are stripped.

If the column reduces on upper floors half the reduction is sawn off each leg of the corner.

When the head is an integral part of the column form the openings for the beams are cut in the panels at the time they are being made up. The opening for a particular size beam should never be the exact dimensions of the beam but always greater, otherwise when the beam sides give slightly under the pressure of the concrete it will be found that the edges of the openings in the panels will be embedded in the concrete and it will be impossible to strip them. For the same reason the beam sides should not be brought flush with the inside of the column sheathing, as in this case the ends of the beam sides will be embedded in the concrete.

One method (*Fig.* 12 (*b*)) is to make the opening $\frac{1}{4}$ in. to $\frac{3}{8}$ in. larger all around than the size of the beam, and bevel the side and bottom edges inwards at about 45 degrees. The beam bottom and sides will butt up against the column sheathing, and the $\frac{1}{4}$-in. clearance will allow for the " give " in the forms.

Another method (*Fig.* 12 (*c*)) is to increase the opening by the thickness of the beam sides and bottom and to nail 2 in. by 2 in. cleats all round flush with the opening.

The main thoughts to bear in mind when building the column head are ease of stripping and allowance for " give " in the forms.

Erection.—If the forms are not too high and heavy, panels can be assembled and bolted up near where they are to be erected and then lifted into place as a unit, this is better than erecting each panel separately and will save some temporary bracing. The column reinforcing is built up into a unit on the floor above and dropped into the column form. If, however, the forms are heavy or have to be lifted high over the reinforcing steel from the column below, then each panel must be erected separately.

Since there is often an inch or so variation in the level of the concrete

at the different columns, especially when the finished floor is put on afterwards, a mark should be given with the instrument on the projecting column or footing steel at, say, 12 in. above the finished floor line, otherwise unevenness or a mistake at one point may be carried up throughout the building. This mark is compared with the mark on the measuring stick, which is also 12 in. up from the floor, and gives the amount to saw off the bottom of the column form before erecting it.

When the form is in place and the bolts tightened, two wedges, preferably hardwood, cut out of 2 in. by 4 in. about 6 in. long, are driven in on each end between the bolts and yokes to bring the ends up tight against the side sheathing.

The use of bolts and washers for tying the panels together is considered standard practice, although there are some excellent patent clamps on the market, which will be mentioned later. Bolts and washers are cheap and can be purchased anywhere, and they are more adaptable to different conditions than other forms of ties and clamps The standard size is ⅝ in. diameter bolt with 3 in. square by ⅛ in. thick wrought-iron washer.

Alignment and Bracing.—It is usual to set exterior columns first accurately and locate the others from these, working towards the centre of the building. After the exterior column forms are set the girder bottoms are nailed on from these to the first row of interior columns, not driving home the nails. This is done in both directions and serves to brace the forms and set them very closely to their correct position. To get the exact positions the measurements are made with a steel tape along the girder bottoms, starting from the building line. This will fix the top of the columns exactly, and they must then be plumbed by suspending a plumb-bob, say, 6 in. out from the sheathing and measuring from this to the sheathing at the bottom. If for any reason the interior forms must be set first they must be temporarily braced ; this can be done by the method described for rectangular columns.

Changing Height of Forms.—When re-using a form it may be necessary to increase its height. This should be done at the bottom, because the pressure there will be greater, requiring a closer spacing of yokes, and also to avoid changing the head if it is part of the column form.

The first extension yoke is put on the bottom of the panels so that the end of the sheathing comes half-way on the yoke andt he joint in the sheathing at the centre of the yoke. To increase the stiffness at the joint vertical 3 in. by 4 in.'s long enough to cover four yokes can be bolted symmetrically across the splice, two to each side, using the yoke bolts (*Fig.* 12 (*d*)).

To decrease the height it is only necessary to saw off a length at the bottom of the form, but if this leaves the lowest yoke too high up an additional yoke must be added about 4 in. up from the bottom.

Re-erection and Reducing Forms.—When using forms again on upper floors it is generally necessary to reduce their size. The column number should be marked on the panels as a means of identification.

After the forms are stripped the panels are carried to the building line and raised to the floor above and placed in their proper location.

If the head is part of the column form the reduction of each panel must be made symmetrically about the centre line, otherwise the beams will be off centre. The two ends are reduced by sawing off a strip from each end of the panel equal to half the total reduction, sawing through both sheathing and yokes. The two side panels are reduced by taking off the two outside boards, one on each end, and replacing them by narrower boards. It is good practice to allow for successive reductions at the time of making up the side panels by having the end boards of widths equal to the successive reductions; it is then only necessary to take off the two outside boards at each reduction (*Fig.* 12 (*f*)).

New bolt holes will be necessary, spaced from the old ones a distance each end equal to half the reduction. It is best to bore all the holes that will be required when the panels are made up. If there are only two stories and one reduction to make, two 2 in. by 4 in., or whatever the reduction requires, can be placed vertically against the end panels between the bolts and yokes and the bolts wedged tight against them.

If the head is independent of the column, or if there are no beams framing into the column, the whole reduction can be made on one end of the panels.

Corner Moulds.—Sharp corners are easily broken when stripping, so it is always advisable to use a triangular bevel strip in the corners; these should be nailed to the sides, not the ends (*Fig.* 12 (*a*)). A rounded corner mould looks well but is much more expensive; if used it should be a part of the end panel sheathing, the wedges bringing it tight against the side sheathing.

Clean-out Holes.—A column form is a convenient receptacle for all the shavings and refuse on the floor, so some provision must be made in all forms for cleaning out the form through a hole at the bottom. This detail must be decided upon and built into the panels when they are made up. The hole should be about 12 in square, depending on the size of the column.

There are two common methods of framing this detail. In the first method the hole is cut out of the sheathing and the loose piece nailed on again with battens, leaving the nails projecting The tongues are cut off so that the loose piece can be slipped back easily, and a long nail should be driven into the batten to form a handle. When ready to clean out the column the loose piece is slipped off and nailed on to the form to prevent it being lost; it is easily put back again.

The other method is to cut out the hole in the sheathing and nail the lowest yoke to the loose piece only, and not to the adjacent boards. A batten across the loose piece will hold it in place until it is required to move it, when the loose piece and yoke are taken off (*Fig.* 12 (*e*)).

Cleaning should be done by a steam or water hose from above.

Rectangular Column Forms (*Fig.* 13).—These are usually exterior columns, the long side being parallel to the building line and remaining

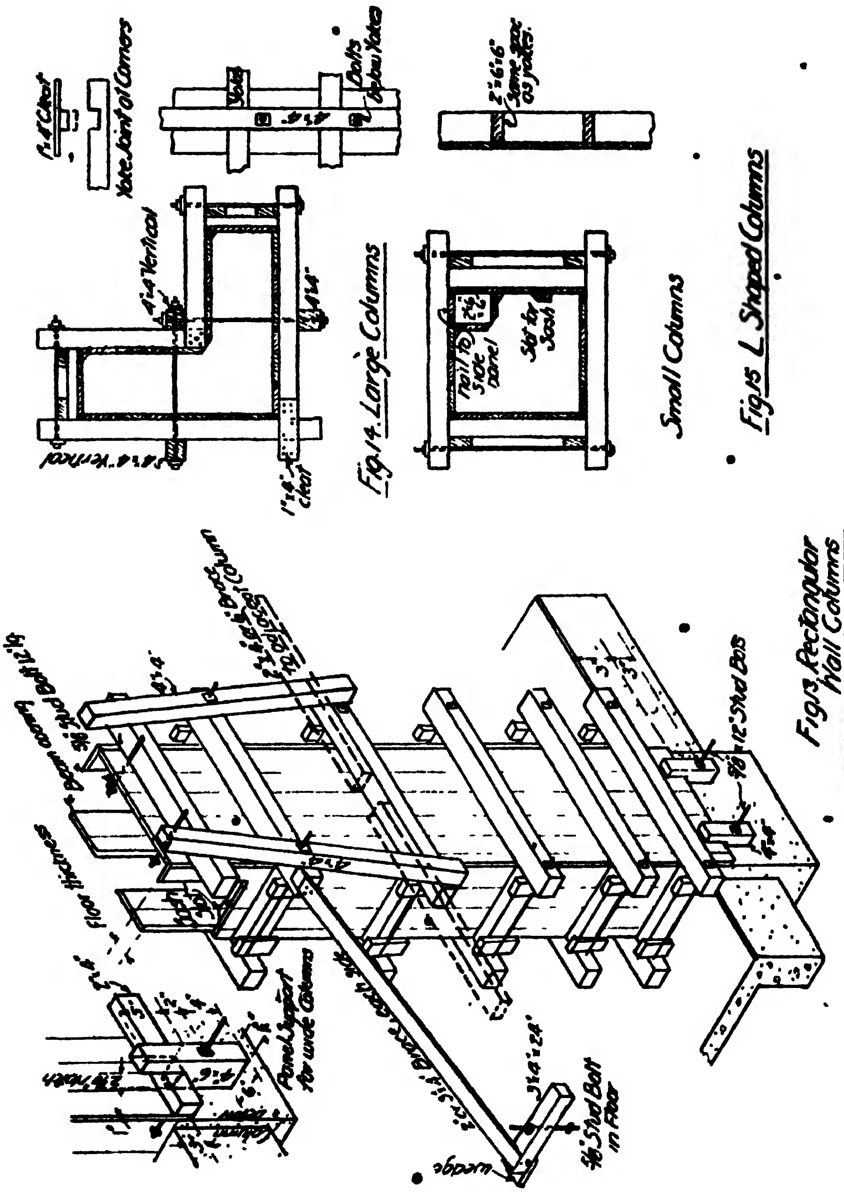


Fig. 13. Rectangular Wall Columns

Fig. 14. Large Columns

Small Columns

Fig. 15 L Shaped Columns

the same throughout all floors, while the short sides or ends often reduce from floor to floor. In general they are built up in panels the same way as square columns, but there will be a few differences since the

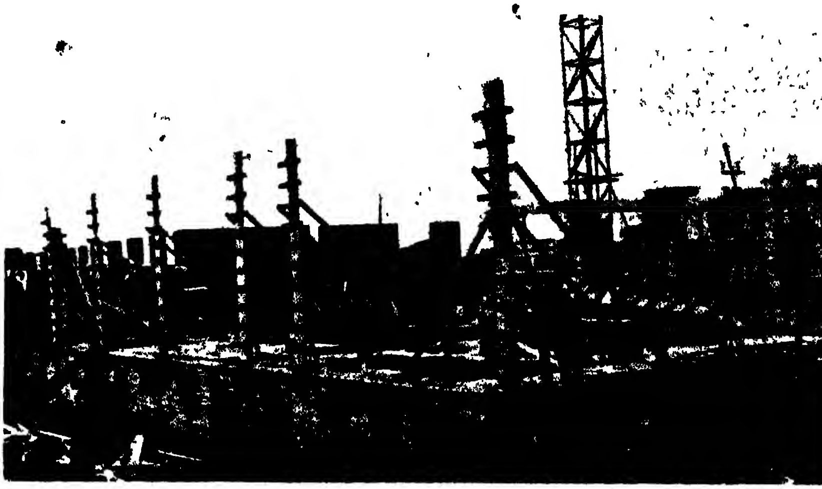


Fig. 16.—Exterior Panels Plumbed and Braced.

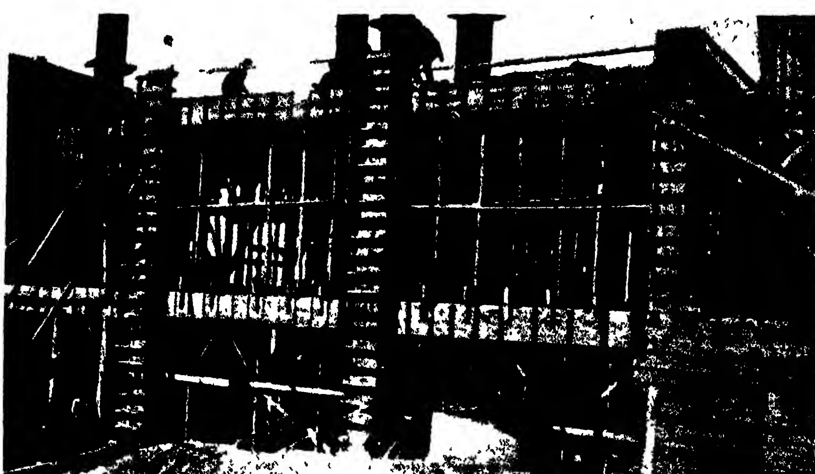


Fig. 17.—Columns 30 Ft. High Vertical Outside Braces.

outside panel will overhang the column below, and the whole form must be carefully braced to prevent tipping over.

The outside panel should be 3 in. longer at the bottom than the inside panel so that it will lap over the column below, and at the top it must reach to the top of the floor slab. The inside panel will be the

same as for an interior column. The two end panels will have the wall beams framing into them, and the sheathing on the outside of the beam must reach to the top of the slab and on the inside to underneath the slab sheathing, unless the head is independent.

For the first story, if the column starts from a footing, the erection will be the same as for an interior column, but if it starts flush with a foundation wall, and for all upper stories, special means must be used to hold the outside panel in place. If it starts from a footing the column will generally be longer than required for the next floor, so when reducing

Fig. 18.—Bracing at Top of Unsupported Panel and Diagonal Bracing to Blocks bolted to Floor

the outside panel is left 3 in. longer than the others; if it starts from a wall this panel is originally made up 3 in. longer.

The outside panel is first erected, and since it will project beyond the edge of the slab it must be supported at the bottom until the other panels are erected and bolted up. This support is provided by embedding two ⅝ in. stud bolts 12 in. long, threaded each end in the concrete, about 6 in. below the top of the wall or column below at the time it is poured. The bolts will project from the concrete 6 in. to 8 in. (with a nut on the end in the concrete) and must be well greased or oiled. The distance apart of the bolts will be about 12 in. less than the width of the column. Over each bolt is slipped a piece of 4 in. by 4 in. about 8 in. to 12 in. long, with a notch cut down one side so that the bottom of the notch will be 3 in. below the top of the column. The depth of the notch should be the thickness of the column sheathing less ⅛ in. The outside panel is slipped into these notches, and when the 4 in. by 4 in.'s are bolted up the bottom will be securely held and the panel drawn tight against the concrete. With very wide panels, say, 3 ft. or more, to be sure that the whole panel is drawn tight against the concrete a 2 in. by 4 in.

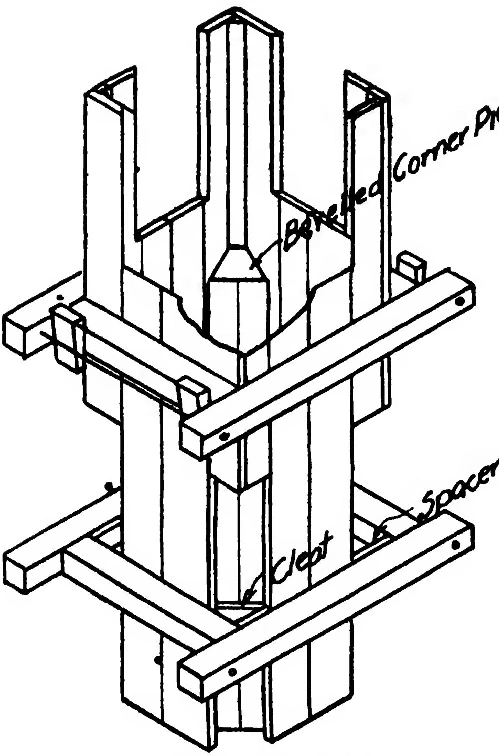


Fig. 19. Octagonal with Square Head

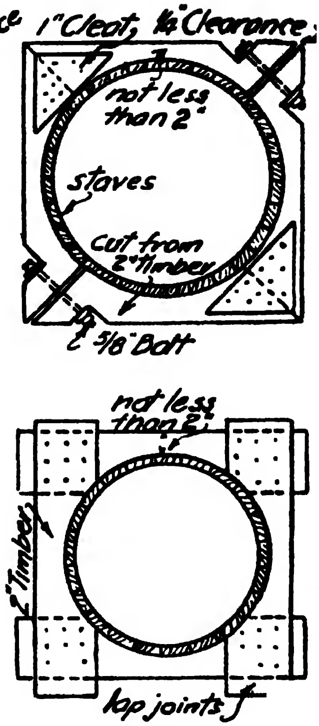


Fig. 21 Round Column

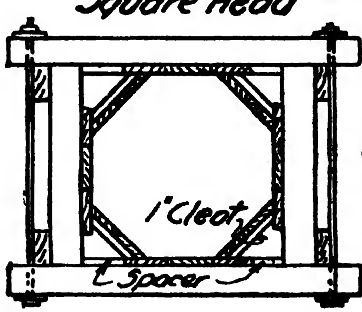


Fig. 20. Octagonal Column

is placed between the notch and sheathing; the depth of the notch will then be $\frac{1}{8}$ in. less than the combined thickness of the 2 in. by 4 in. and sheathing.

Before the panel is erected a 2 in. or 3 in. by 4 in. is nailed on to the end of a yoke near the top, one on each end, and the panel is held by these braces while being put in place. When the panel is plumbed the other ends of these braces are nailed to wooden blocks, fastened to the floor, either to the top or side of the block. Provision should always be made when pouring the floor to provide means for holding these blocks. It is best done by leaving stud bolts at the required places in the concrete, with nut down, and well greased and projecting 6 in. above the floor. Over these bolts short pieces of 3 in. or 4 in. by 4 in. can be slipped and the nuts screwed tight, a wedge driven in between the timber and the floor will prevent it turning.

Another method is to twist some wires around the reinforcing steel and leave the ends projecting to be twisted around the 4 in. by 4 in. Any stud bolts left in the concrete, either in floors or columns, if well greased, are easily twisted out when finished with and the holes filled with mortar so that they will not show.

When the outside panel is plumbed and braced the two end panels and the interior side panel are erected and all bolted up. Two-inch by 4 in. or 6 in. braces, depending on the depth of yoke, should extend from column to column along the outside to keep the forms in line in that direction.

The outside panel of the column head must next be braced, since there is no beam entering it. The top pair of yokes that can be bolted together will be below the deepest beam framing into the head, and since there will be one outside yoke above these it must be supported by other means. This is done by bolting to the top pair of bolts two vertical pieces of 4 in. by 4 in. long enough to bear on the three top yokes. Instead of using the yoke bolts two short bolts can be used, one on each end, making this yoke about a foot longer to allow for them; by this means when stripping the form the two vertical pieces will always be attached to the panel and so will not become lost. This same method is used for interior columns when one panel of the head is unsupported, and can be used for end panels by placing the vertical pieces between the wedges and the yokes.

It is usually necessary to attach steel sashes to exterior columns, and to do this a slot must be left in the side of the column into which the sash is slipped and grouted. These slots are made by nailing a 2 in. by 4 in., bevelled at 45 degrees on the inside edge, to the end panels for the full height of the sash. It may be necessary to bond a partition wall into the column, in which case a similar 2 in. by 4 in., but bevelled on each edge, is nailed to the column form on the centre line of the partition.

L-shape Columns (*Figs.* 14 and 15).—These are almost always corner columns. If the difference in width between the short and long sides is not over 12 in. it is best to build up a square column form with

sides equal to the long side and then block out for the " L " in one corner. The corner piece is made by nailing sheathing on to 2 in. timber spaced the same distances apart as the yokes. It is built as a unit and then nailed to one side panel.

Corner columns are often very large for architectural effect, and another method must then be used. There will be six panels to a complete form, the yokes being spaced similarly on each. The yokes on the two long and two short sides are notched over each other at the corners, those on the long sides overlapping, with a short piece of 1 in. by 6 in. nailed over the joint. The two ends are built and bolted in the ordinary way. As the sides will be long, one or two intermediate bolts will be necessary. These bolts should pass through vertical 4 in. by 4 in. or 6 in.'s outside the yokes to stiffen the form, one of them being at the inside corner. The panels with the notches cut in the top of the yokes must, of course, be erected first. The panels are supported and braced in the same manner as rectangular columns, only on two faces instead of one.

Octagonal Columns (*Fig.* 20) —An octagonal form is a modification of a form for a square column.

Four panels are made up as for a square column, but the sheathing need only be 4 in. wider than the side of the octagon, except that at each side yoke the sheathing is blocked out with spacers to make the total width equal the diameter of the column plus twice the thickness of the sheathing in order to give bearing for the end yokes. The remaining four sides are single boards of the required width or two or more boards cleated together, with the sides bevelled at 45 degrees. The loose sides are then nailed in place at the yokes just sufficiently to keep them in place. The side of an octagon is 0·414 times the diameter.

To reduce the form it is necessary to saw a strip off one edge of the loose sides and both ends of the end panel yokes and to reduce the width of the spacers.

The head of an octagonal column is often square to receive the beams. In this case the head is fully sheathed as for a square column and the four corners are filled in with pieces of sheathing, or, if small, with solid pieces cut to fit (*Fig.* 19)

Round Columns (*Fig.* 21).—Wooden forms for round columns are seldom used unless there are only a few columns, since steel forms, which can often be rented, are cheaper.

In order to be able to strip easily and to use the forms over again the following method can be used. The yokes are cut out of 2 in. timber, the width of the boards depending on the diameter of the column; a minimum of 2 in. should be left after cutting out the circle. The yokes are cut as in *Fig.* 21, the diameter being the diameter of the column plus twice the thickness of the sheathing. The joints in the yokes are made on two diameters at right angles; two opposite joints are nailed together with 1 in. cleats and the pair at right angles are bolted together, notches being cut in the yokes so that the bolts will have square bearings.

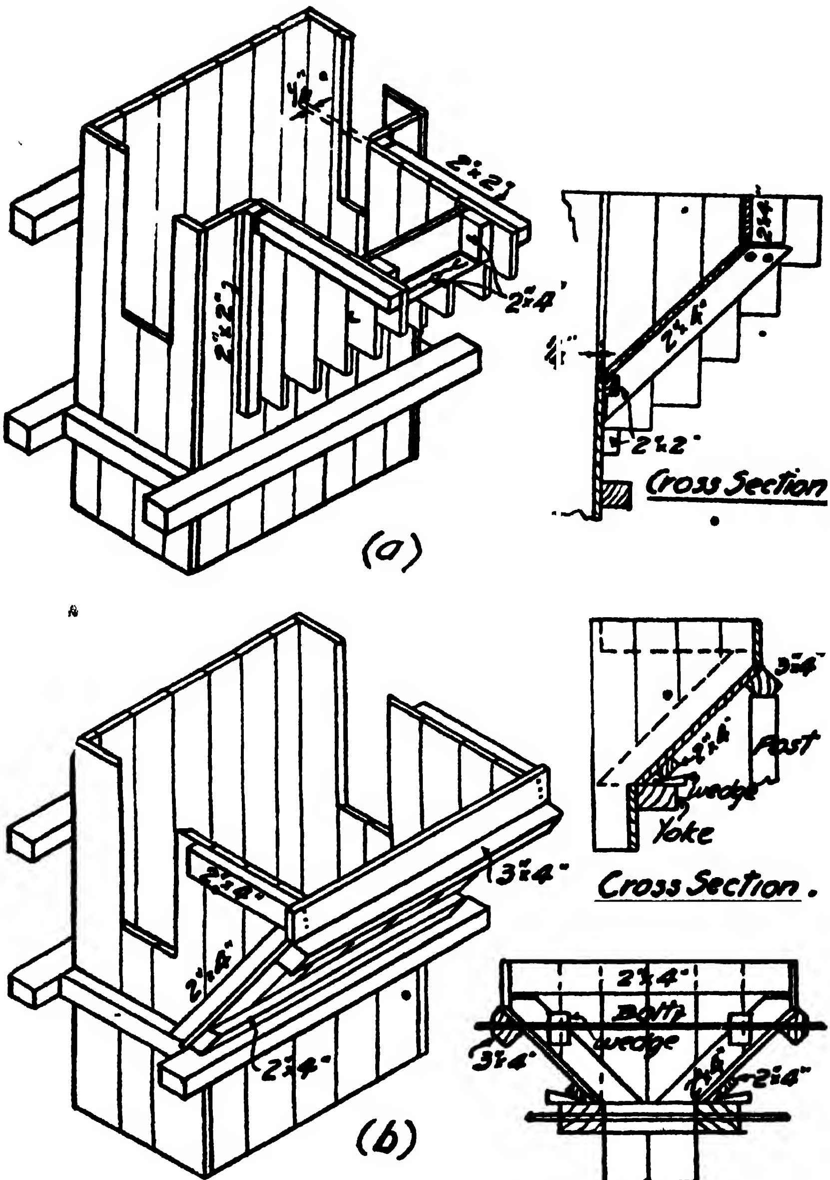


Fig.22. Column Brackets

Double Bracket

There should be at least ¼ in. clearance at the bolted joints so that the joints can be drawn tight.

The sheathing will have to be stave-shaped to the desired radius unless the column is very large, when 1 in. by 4 in.'s can be bent around the circle and any sharp edges afterwards rubbed off. The spacing of the yokes will depend on the safe span of the sheathing, which can be obtained from the tables.

Instead of the two butt joints and battens, a joint with less strength, but sufficiently strong for small columns, is made by overlapping the yokes and securely nailing them together. If the form is only to be used once the bolts can be dispensed with and all joints made butt or overlapping. These forms cannot be reduced without making new yokes; new staves will not be necessary unless the reduction is very large.

Very High Columns.—Columns of more than the ordinary heights up to 12 ft. should be braced or stiffened at intermediate points to give them sufficient rigidity. Long outside columns should have vertical 3 in. by 4 in.'s on the outside yokes, the bolts being long enough to pass through them. Inside columns can be stiffened in the same way or by diagonal braces.

Stripping Forms.—Columns are stripped from two to three days after pouring. The concrete should be sufficiently hard so that the corners will not be broken off in stripping, but should not be so hard that it cannot easily be rubbed smooth. The common test is to strip when the finger-nail will not dent the concrete, or when the concrete rings to a blow of a hammer.

To strip, remove the wedges first, then the bolts, and pry out the bottom of the side panels until they are free of the beams, when they can be let down. Stripping should not start at the top of the column but at the bottom. Wherever possible pry on timber and not on concrete.

Brackets (*Fig.* 22).—Brackets are often built as an integral part of the column form, as in mushroom slab construction, and when brackets are required to carry a crane girder. Two methods of construction are shown in *Fig.* 12; (*a*) is more suitable for small brackets and (*b*) for larger brackets the width of the column.

In (*a*) the bottom is carried on 2 in. by 4 in.'s nailed to the sides, which are nailed to the cleats around the opening. The sides must be held by a bolt or wire to prevent spreading.

In (*b*) the sides are cut to the slope, with 2 in. by 4 in.'s nailed along the top and slope. The bottom is a separate panel of sheathing nailed to a 2 in. by 4 in. at the bottom, which is wedged up from the yoke against the sides, and to a 3 in. by 4 in. at the top, which is supported by a post at each end or bolted through to the column yoke at the back. The posts may be carried on a yoke, in which case they should be braced from the side forms. If there is a double bracket the 3 in. by 4 in.'s at the top are bolted together. Note that there should be ¼ in. clearance between the bracket and column sheathing.

Cost of Column Forms.—Column forms require 0·2 to 0·3 cubic feet of timber per square foot of contact, and an average value can be used for estimating. The larger the column the less the timber required per sq. ft. It is usual to allow sufficient timber to form all the columns on the lowest floor and to use the forms again on the upper floors. The timber required will therefore be the surface area of the lowest story columns multiplied by the above unit. If an upper story is higher than the lowest, this quantity should be multiplied by the ratio of the heights to allow for splicing out.

The forms should be good for use at least six times, after which some allowance should be made for renewals, say, 15 per cent. per floor.

Making up panels and erection will require about 1 labourer to 2 carpenters, and stripping will require 2 labourers to 1 carpenter. The stripping may be done by labourers only, but a carpenter will take more care of the forms and it will pay to employ one.

To complete 100 sq ft of interior square column forms will require 4 hours carpenters' time and 2 hours labourers time making the panels; 8 hours carpenters' time and 4 hours labourers' time for erection, aligning, and plumbing, and 1½ hours carpenters' time and 3 hours labourers' time for stripping Cleaning forms and hoisting to next floor will each require about 1 hour labourers' time

Cost of 100 sq ft of forms, 1st erection -
- 13½ hours carpenter at ......
- 9 ,, labourer at ......
- Timber, 100 × ·25 = 25 cu ft. at ......

Cost of 100 sq. ft of forms, each floor above
- 9½ hours carpenter at ......
- 9 ,, labourer at ......
- No timber will be required.

Exterior columns will require about 25 per cent. more time to erect.

Octagonal columns will cost about 50 per cent. more than square, and round columns about 100 per cent more per sq ft.

Brackets will cost about 50 per cent more per sq. ft. than square columns.

CHAPTER VIII.

WALL FORMS.

The same general methods of construction will apply to any kind of wall and whatever the height and location, but it is convenient to consider separately basement and partition walls, low spandrel walls, very high walls or piers, retaining walls, and curved walls, as each kind will have a few distinctive features.

The first thoughts will be whether to build the forms in place, assembling and erection being done in one operation, or in panels where assembly is done at a central point and only erection is done in place; whether to build sufficient forms for the whole wall, or to use the forms a number of times over, and in the latter case whether to raise the forms vertically or to move them horizontally These questions have to be decided for each individual job, but a few general statements can be made.

It is always well to build sufficient forms for one full day's pour, say, from 40 cu. yds. up, depending upon the size of the mixer. If a wall can be poured in 7 hours there will be at least 1 hour each day left, most of which time will be wasted as the last hour cannot be used efficiently.

On a large job it will probably be necessary to build sufficient forms for two or three days' pour so that the carpenters can always keep ahead of the concreting gang, in this case the forms should be made up in panels for quick stripping and erection

Small walls that can be poured in one or two days are generally cheaper formed in place, as panel forms will only save money when they can be used several times.

Forms for spandrel walls up to sill line can be used several times, and so they should be made up in panels.

Basement walls and any walls up to about 12 ft. high are generally formed the full height. Above this height they become unwieldy and more difficult to brace, and so are sometimes formed halfway up and the forms raised vertically. It is easier, however, to make a good vertical joint than a good horizontal joint; and cheaper in labour to move forms horizontally than vertically whenever possible, since they are easier to handle. Also, structurally a vertical joint is better than a horizontal one. Very high walls, gravity retaining walls, piers and dams are often built with panel forms 6 ft. to 10 ft. high, moved vertically. This is not always permitted, however, for engineering reasons, and it is often necessary to form the wall the full height.

Very high wall forms can be moved horizontally, but special handling

equipment is then necessary, and they are generally of steel, not timber. "Moving," or "creeping," forms will be described in another chapter. For all walls built in place, and also for panels if not used more than two or three times over, 1-in. sheathing is thick enough. With several usages it will pay to use 1¼-in. or 1½-in. sheathing. If particularly good lines are required it is better to use the thicker sheathing.

Basement and Partition Walls.—These will cover all ordinary walls in a building, whether in the foundations or as partitions. They will be from 8 ft. to 12 ft. high, and 2-in. by 4-in. studs are generally used at from 14 in. to 16 in. on centre with 1-in. sheathing.

Note from the design tables that the stud spacing depends upon the allowable span of the sheathing. For a wall 10 ft. high a 3-in. by 4-in. stud can only be spaced 1 in. farther apart than a 2 in. by 4 in. while the same number of wales or ties will be required, and so since a 3 in. by 4 in. will cost considerably more than a 2 in. by 4 in. there would ordinarily be no economy in using the heavier stud. However, if there are to be no exterior braces the heavier stud would be better as it would give a stiffer form.

Note also that the allowable span of the sheathing is based on a deflection of ⅛-in. When the deflection does not matter, as when a wall is covered up, the studs may be spaced farther apart, but the ties should be closer.

Studs need not be cut off at the top of the wall but can be used in random lengths to avoid waste In all form building an inexperienced man may spend a lot of money cutting up his material to exact lengths resulting in neat but wasteful forms.

If the form is to be built in place, which is the cheapest method for small walls, it should be built as lightly as possible, consistent with strength. The number of nails used should be kept down to a minimum, just sufficient to keep the different members of the forms in place. Nails are seldom used for strength, and they increase the cost of stripping and lessen the salvage value of the timber.

The studs are first set to line all the way through with a board nailed on top and bottom to hold them together, and temporarily braced with 1-in. by 6-in. braces. Lines should always be set an inch or two away from the actual face of the wall so that the forms will not interfere with them. The lowest board should be nailed on level, otherwise the sheathing will tend to run up or down hill, and if the foundation is stepped or uneven the special widths of boards are put on last, not first. The sheathing is nailed on lightly, breaking joints preferably at the studs and the tongue tapped well into the groove so as to get a tight form.

If the wall is narrow and there is much reinforcing steel to place, one side should be completed first, plumbed and braced, and the other side set from this by means of 1-in. by 2-in. spacers cut to the exact width of the wall. The spacers should be about 3 ft. apart vertically and at alternate rows of studs, and are knocked out as the concrete reaches them. If the wall is wide enough for a man to work inside to

place the steel both sides may be brought up together and less temporary bracing will be required since the studs can be tied across at the top with short pieces of board.

Only one side of a wall should be accurately set to line, plumbed, and braced, since the other side will be set automatically by the spacers. The side to set first depends usually on which is the most important face of the wall, generally the outside if exposed to view and the inside if below ground, also ease of bracing enters into consideration. A good deal of time can be wasted in trying to set and plumb each side independently.

When the form is completed the temporary braces should be replaced by 2-in. by 4-in. or 3-in. by 4-in. braces, as in *Fig.* 24, about 10 ft. apart, the distance depending mainly on how stiff the forms are built. If the wall is low, 1-in. by 6-in. braces will be sufficient, as in *Fig.* 23. Low and unimportant walls can be built without wales, but above 6 ft. high there is always danger of the form twisting and getting out of line from the impact of the concrete if wales are not used Also wales will transfer the pressure from one stud to another if a tie is loose or breaks. Wales should only be nailed on sufficiently to hold them in place.

When wales are used it is possible with care to do without exterior braces, even with very high walls, but a few are always advisable. The braces should be wedged against the wales, and are very useful if the form begins to get out of line since the wedges can be loosened or tightened to bring the form to line again.

The ties can be inserted before plumbing the forms but should not be tightened up until afterwards, if done beforehand some will become slack and others too tight and they will have to be gone over again.

Ties.—Whether to use wire ties or bolts is always an open question. For very accurate and important work bolts are undoubtedly better, but for ordinary walls in buildings it is almost universal practice to use wires It is more a question of practical knowledge than theory whether wires are strong enough and how many should be used. It depends mainly on the temperature and the speed of pouring, since in warm weather and with slow pouring the concrete will set fast enough to support itself to some extent and so relieve the pressure on the ties.

If a wall 12 ft. high, for instance, is poured in 8 hours, at least 4 ft. to 6 ft. of it will be set sufficiently by the time pouring is finished to support itself and the wet concrete above to a large extent so that the pressure in the lower ties will be very much reduced from what it would be if the wall were poured in 2 hours. It has been seen in the previous chapter, Design 13, that for a wall 8 ft. high two double strands of No. 9 wire, or 4 wires, are required on the lowest wale, assuming maximum pressure.

In general, for walls up to 12 ft. high, with 1-in. sheathing and every stud tied, two double strands of No. 9 wire should be used on wales in the lower half of the wall and one double strand on wales in the upper

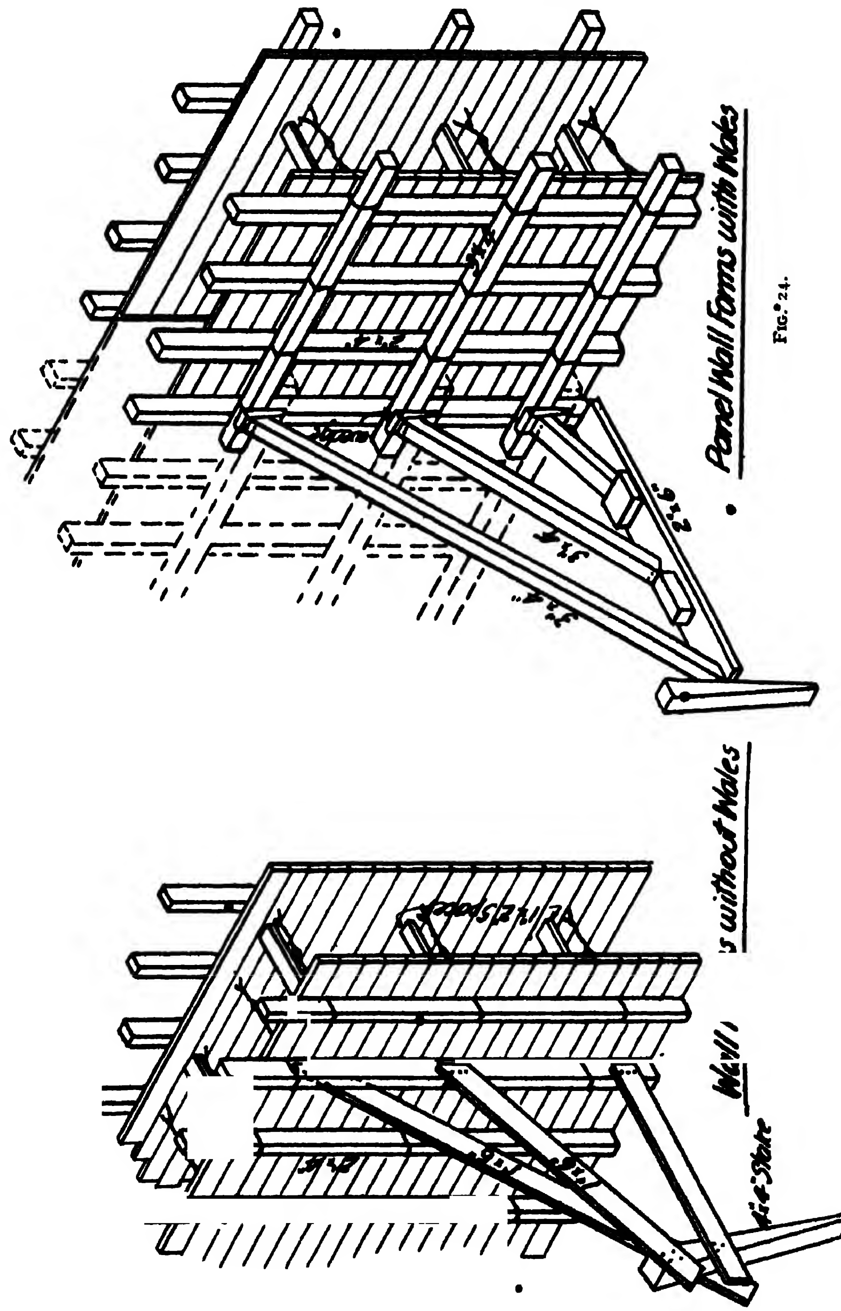


FIG. 24.

half of the wall. With thicker sheathing, and when only alternate studs are tied, bolts should be used and their size calculated. When wales are used the wires can pass around stud and wale or around the wale only. The holes are bored from one side of the wall only, using an extension bit to bore through the far side.

The wires should be twisted up tight inside the wall, usually with the claw end of a hammer, but not so tight that they are on the verge of breaking, and there should be a spacer just above each wire to draw the forms up against. The spacer should be above the wires, as they are commonly used for walking on and placing them above prevents walking on the wires. The wires should be hammered flat against the wales or studs, before tightening up, or wedges driven in after tightening, otherwise the slack will be taken up under pressure and the form will give slightly. Two men should work together on the wires, one each side the wall.

Bolts are not used with single studs and no wales, but if a stud is made up of two pieces the bolt is placed between them, bearing on a short piece of timber. When wales are used the bolt is placed through the wale only, close to the stud, to save boring through the stud.

If a contractor is doing a great deal of concrete work he will generally find plain round rods and patent clamps more economical than bolts, since a smaller size can be used in the first place and they can be ordered long enough so that they can be used many times over in walls or columns. They are more easily drawn than bolts, and if they have to be left in the wall their loss is less. Bolts, to be adaptable, must have long threads, because of the different thicknesses of wall, or else several lengths must be carried, while rods 3 ft. or 4 ft long can be used almost anywhere The initial cost of clamps, however, does not warrant their use unless they can be used several times over.

Wires are cut off a little within the surface of the concrete, so that they will be covered when finishing and there will be no danger of rust spots. Bolts and rods are more often drawn than left in, so they should be greased before pouring. They should be withdrawn within two days after pouring, before the concrete has set too hard to make it impossible or expensive. Rod pullers can be bought for this purpose.

There are many devices, mostly patented, for enabling the ends of a tie to be unscrewed leaving the centre of the tie in the wall, and as their main advantage is that a hole is not left right through the wall it may be necessary to use them when a wall has to be absolutely watertight. A simple device of this kind consists of a rod a little shorter than the width of the wall, threaded each end, and fitted with unions into which short pieces of rod are screwed. These end pieces are easily screwed out and the holes plugged (*Fig.* 29). To withdraw a bolt easily an iron pipe-sleeve a little shorter than the width of the wall and fitted each end with a wooden washer can be slipped over the bolt.

The disadvantage of special devices is that they generally have to be ordered for a certain width of wall and so are not adaptable and

also a part will be lost in the wall. A greased rod, and even an ungreased rod, will easily pull out of a green wall if given a twist to start it and can be used again many times.

Panel Construction.—On a large job where wall forms can be used several times over it will be best to build them in panels. This has also the advantage that the panels can be made up in advance, so saving time in erection. Their size will depend on the facilities for handling them; 8 ft. by 10 ft. is a convenient size, weighing about 550 lbs., and can be handled by four to six men.

Wales are generally put on during erection and the holes bored at that time, and the wales should be long enough to overlap the end stud of the adjoining panel. The ends of the sheathing must, of course, be cut off square; at one end of the panel there will be no stud, and at the other end the sheathing will finish at the centre of the stud, so that the next panel can join on to the same stud (*Fig.* 24). One side of the wall will be set to line first, plumbed, and braced. The spacers will be lightly tacked on to this form and the other side up-ended against the spacers, bolts or wires inserted and drawn up tight against the spacers.

As these forms will have to withstand much more handling than when built in place, 1¼-in sheathing and 3-in or 4-in. by 4-in. studs are better than the lighter sizes. The studs in panel construction should preferably be of the same length as the height of the panel. Wall panels can sometimes be used again in the floor construction, and can be built in the first place with that in mind. The panels should be oiled before being erected.

Double Studs and Wales.—For light work some contractors use double studs consisting of two pieces of 1 in. or 1¼ in by 4 in. instead of a solid 2 in. by 4 in. The two pieces are separated by 1-in. thick blocks about 3 ft. apart, forming a slot through which the bolt is passed (*Fig.* 28). A short piece of 2 in. by 4 in. must be slipped over each end of the bolt to give bearing for the washers. Wires cannot be used with double studs. The forms are made in panels, the sheathing being nailed together with 1-in. by 4-in. battens at the same spacing as the studs, which in this case are not nailed to the sheathing. The panels are erected, plumbed, and braced, the holes bored, and the built-up studs placed in position and bolted up tight against the spacers, spacing the studs between the battens.

Although there may be a small saving in timber cost, the labour will cost much more than with solid studs as there are so many more pieces to handle. There will also be less salvage with the lighter timber, so that using double studs is doubtful economy. There may be some economy, however, in using double wales, especially when a heavy size is required, such as a 4 in. by 6 in., when two 2 in. by 6 in. may be much cheaper in material and the labour cost will be little more. The bolts, as before, are inserted between the two pieces, and wood bearing blocks need not be used if the waling pieces are 2 in. thick or over (*Fig.* 29).

Column Offsets.—Basement walls often have offsets where the columns are carried, or a thin partition wall may be poured with the

column giving a projection on each side. The first can be built as in *Fig.* 30. The face of the projection has the sheathing extended to carry the stop battens to which the sides are nailed.

In the second case an ordinary column form is built with the end panels cut to leave an opening equal to the thickness of the wall plus twice the thickness of wall sheathing. The wall sheathing should stop

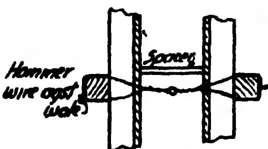


Fig 25– Wire Tie

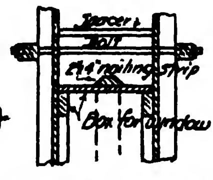


Fig 26 Window Opening

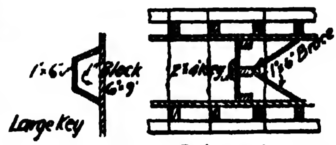


Fig.27 Bulkhead and Key

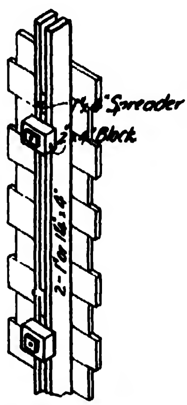


Fig 28– Double Stud

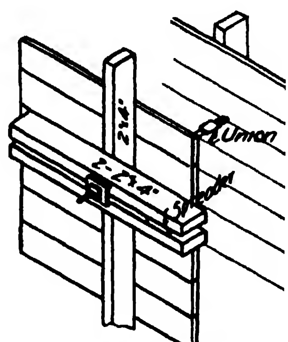


Fig 29 - Double Wale

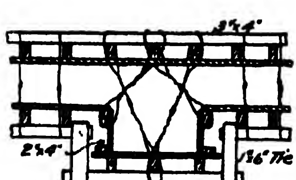


Fig. 30– Column Projection

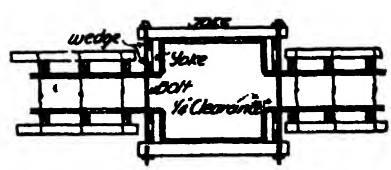


Fig. 31– Framing Wall into Column

about ¼ in. from the inside of the column sheathing, and should have bevelled edges to allow for give in the column forms (*Fig.* 31).

Bulkheads and Keys.—When pouring a wall in vertical sections a bulkhead has to be put in at the end of each section and a key left for the new concrete. This is constructed as in *Fig.* 27. If the wall is wider than the allowable span of the sheathing a centre stud must be used and tied diagonally to the side studs or braced from the outside.

For a heavier wall a larger key should be made of 1-in. boards nailed to 1-in. blocks bevelled two sides (*Fig.* 27).

Window Openings.—When there are basement windows in a wall a four-sided box is built the size of the window opening and nailed into the form (*Fig.* 26). For small walls the box usually consists of 2-in. plank, and for larger walls of 1-in. sheathing, on 2-in. by 4-in. frames. Around the outside of the box must be lightly nailed a 2-in. by 4-in. nailing strip to which to nail the wood window frame, or if steel sashes are used to form a groove into which to grout the sash. With steel sashes the window sills are usually poured after the sashes are set, so that the size of the box must include the depth of the sill. The nailing strip must be nailed on lightly so that it will remain in the concrete after stripping the box. To make stripping easier the box is often sawn through the middle of each side and the halves joined with battens, which are taken off before stripping.

Corner Bracing.—The weakest part of a wall form is usually at the corners; there being no studs directly opposite the corner one, tying or bracing this is often forgotten. Wales should overlap at the corners and be spiked together, and the outside corner studs should be tied diagonally across to the inside corner stud either separately or with wires around both studs

Single Wall Forms.—A wall poured against an existing wall or against the bank of an excavation needs only one form, and this must be braced externally. If there is anything close to brace to horizontal braces are best, but usually they have to be taken diagonally down to the ground where they are held by stakes. In order to be able to adjust and plumb the form, wedges are used between the brace and wales, driving a nail through both to prevent slipping (*Fig.* 24). If there are any signs of the wall going out of plumb the nails are withdrawn, the wedge driven up, and the nails replaced.

If no wales are used, 1 in. by 6 in. for low walls and 2 in. by 4 in. for high walls are used as braces at each stud and are nailed to stakes (*Fig.* 23). With wales the braces are generally 3 in. or 4 in. by 4 in. and they can take bearing on a 2-in. plank or 4 in. by 4 in. laid on the ground with a stake at the end (*Fig.* 24). If it is necessary to brace to a finished floor the 4 in. by 4 in. can be held with stud bolts as for columns.

Low Spandrel Walls.—These walls span from column to column and are only sill high. As there are generally several walls of the same height and length the forms are made up in panels of 1 in., or preferably 1¼ in., sheathing on 2-in. by 4-in. studs. They are built after the columns are stripped, so that the main problem is the method of bracing them.

To hold the bottom of the outside form the stud bolts left in the column 6 in. below the floor line can be utilised. Two 2 in. by 6 in. separated by 1-in. blocks can be slipped over these bolts and bolted tight to the column. These will hold the studs, which must be about 8 in. longer than the sheathing. If the wall is flush with the lintel the

Fig. 32.—PANEL WALL FORMS USING INTERMEDIATE BATTENS AND PATENT WIRE TIES.

Fig 33.—WALL PANELS, 18 FT HIGH; THREE PANELS VERTICALLY

Fig. 34.—DIFFICULT WALL FORMS FOR AN EXPOSED CONCRETE EXTERIOR.

sheathing should extend an inch or so below the floor. If the wall is set back from the lintel it may be necessary to wedge between the studs and wale (*Fig.* 36*a*).

Another method is to nail a 2 in. by 6 in. on to the bottom of the studs and to hold it by short pieces of 4 in. by 4 in. over the bolts (*Fig.* 36*b*). At the top a similar wale is bolted through close to the columns to a 4 in. by 4 in. on the other side of the columns, long enough to hold the first two studs. The inside form is held at the bottom by wires or bolts in the ordinary way, and the top by nailing short pieces of 1 in. by 6 in. across the studs or by wires.

Sills are usually poured after the walls are stripped and the sash set, but if the sash is set before the walls are poured the sills and walls

FIG. 35.—WALL FORMS FOR ORNAMENTAL EXPOSED CONCRETE EXTERIOR.

can be poured together. To form the sill projection the studs can be blocked out from the sheathing 1 in. or 2 in. as required, or notches can be cut in the studs (*Fig.* 36).

Very High Walls and Piers.—If a high wall is to be formed to the top the methods will be the same as already described. Timber sizes need not necessarily be heavier if care is taken, but since it is difficult to brace high walls from the outside heavier timber will give a stiffer form, so 1¼-in. sheathing and 4-in. by 4-in. studs are preferable. Wales should always be used, and bolts or rods instead of wire, except perhaps near the top.

High walls and piers are often built by raising the forms vertically in panels if horizontal joints are not objectionable. The panels are made the same as when they are moved horizontally, but they should be about 6 in. higher than the height of pour in order to lap over the concrete

already poured. One lift is all that can be managed with hand-power, using a hand derrick. With derricks, cranes, cableways, etc., very high lifts can be made. A panel 8 ft. by 10 ft. is convenient to handle, but with power handling larger panels may be used.

To support the bottom of the panel after it is raised, stud bolts or rods should be left in the wall on each side at each pour, 6 ft. to 8 ft. apart and about 9 in. down from the top of the pour (*Fig.* 37). To these is bolted a 4 in. or 6 in. by 6 in., which acts as a support for the panels. To hold them temporarily while being plumbed, wires can be embedded in the concrete, twisted around the reinforcing steel if possible, and sufficiently long to reach to about the centre of the studs. The first wale and bolts should be just above the concrete, so that the forms can be drawn in tight against the concrete to avoid leaving a ridge at the joint. Using spacers in the wall there will be little difficulty in plumbing the forms. High walls built with moving forms will be mentioned later

Retaining Walls.—Plain concrete walls will be built by one of the methods described, but reinforced concrete retaining walls may have a few special features

With cantilever walls there is often a fillet or bracket at the bottom of the back of the wall (*Fig.* 38). The form for this is built as a separate panel with the same spacing of studs as for the wall itself. As there will be upward pressure on this panel it must be well-anchored down, generally by wiring to the reinforcing steel. The wall panel is set on top of the sloping panel and the studs of each nailed together. There should be a spacer at the bottom of the wall panel. A diagonal 1-in. by 6-in brace is nailed from the sloping to the vertical studs to help support and stiffen the wall form. The first pair of wales will be at the junction of the sloping and vertical studs, but if this is more than about 2 ft. high the bottom of the studs must be held by wales bolted through the wall or braced from the outside. As the back of the wall will usually be on a batter, the spacers will vary in length The top and bottom spacers must be cut to the exact width of the wall and are set first spacing the top and bottom of the forms. The intermediate spacers are cut approximately to suit the width and are wedged into the forms to hold the panels to line.

Counterfort walls present no particular difficulty. The sloping back of the counterfort is built similarly to a bulkhead. For easy stripping it should be built as a panel with 1-in. by 4-in. battens down the centre and only lightly nailed to the stop battens on the sides, as the pressure of the concrete will keep it in place. The sheathing of the sides of the counterforts should not be cut to the slope, but should be of random lengths to save material. Since counterforts are usually about 8 ft. apart they are easily braced against each other with diagonal braces butting against a stop block nailed to a 2-in. plank or a 4 in. by 4 in. which holds the lowest wale (*Fig.* 39). When the counterforts are short it is usually better to run the sheathing vertically. The braces should

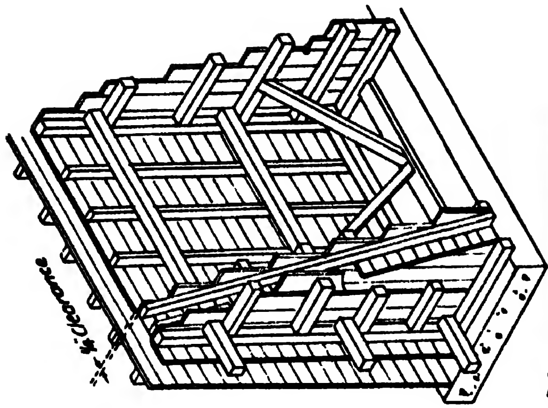


Fig 39 – Counterfort Ret'g Wall

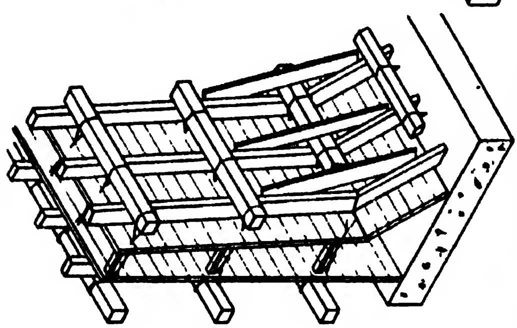


Fig 38 – Cantilever Ret'g Wall

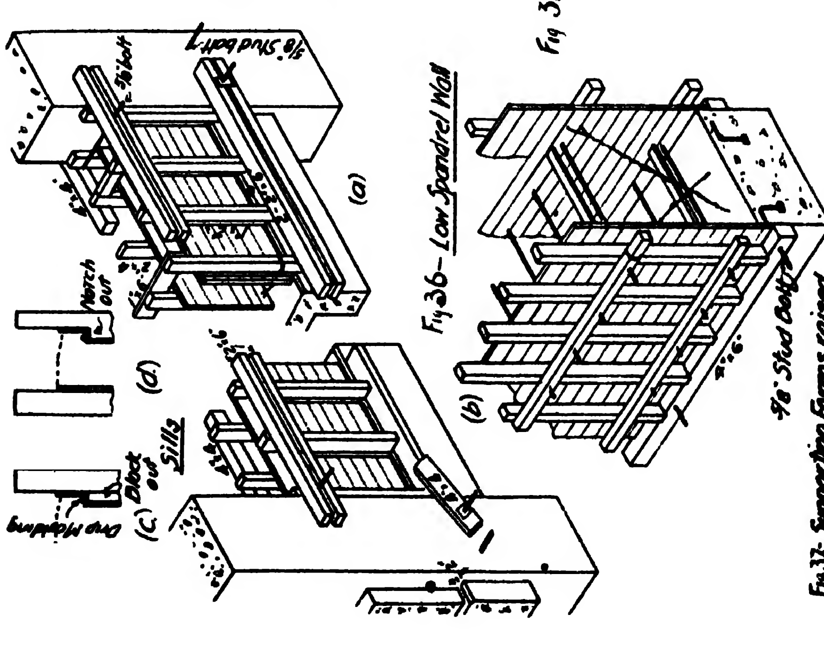


Fig 36 – Low Spandrel Wall

Fig 37 – Supporting Forms raised vertically.

not be horizontal between counterforts unless the concrete is brought up in each one at the same rate. As the counterforts will be stripped first the wall sheathing should overlap the counterfort sheathing (*Fig.* 39).

Curved Walls.– These will have vertical sheathing and horizontal yokes like a circular column (*Fig.* 40). The curve should be laid out on the ground by driving in a stake at the centre and marking out the radii with a lath nailed to the stake and with a hole at the other end for a pencil. Along the lath is measured the outside radius plus the sheathing thickness and the inside radius less the sheathing thickness. Along the circumference are loosely placed 2-in. planks the width depending on the radius, lapping 6 in. to 12 in. The lath is swung around over the planks, which are adjusted until the laps are about equal and the depth of cut at the ends the same, when they are nailed lightly together and the curve marked out on them. They are then taken apart and the curves cut out. When put together again with the same amount of lap they will give the correct radius.

When possible it will save timber to cut both inside and outside yokes from the same plank and to do this it will pay to use wider and shorter plank. The inside yokes should be marked out first and then the planks taken apart and moved farther out until the greater outside curve coincides with the inner curve at the centre of the yokes. The outside yokes must be longer than the inside yokes because of the greater curvature if each are lapped the same amount, the ratio of the lengths being approximately the same as the ratio of the outside and inside radii. The inside yokes should be cut back at each end to avoid feather edges. Sometimes only one plank is marked out in this way and used as a template for cutting the others, but if there is a slight error in the template this will be repeated in all the yokes and they may not fit together so well.

Instead of using 2-in. planks two thicknesses of 1-in. boards can be used nailed together, with the splice in one board at the centre of the other board as shown in the dotted lines in *Fig.* 40. This is more suitable for inside than outside yokes. The least depth of outside yoke at the centre should be not less than 4 in., nor the end depth of inside yokes less than 4 in. at the centre of the lap.

When the yokes are cut out they are nailed together at the laps, the radius being checked again before driving home the nails. The sheathing is nailed on either in place or in panel form. The sharper the curve the narrower must be the sheathing.

The spacing of the yokes depends on the allowable span of the sheathing (Table 2). When erected the forms are wired across at the laps of the yokes and at intermediate points so that the wires are not over 30 in. apart, depending on the height of the wall. If the wall is high, vertical wales should be used at the ties to stiffen the form.

With very flat curves it is often possible to spring the boards horizontally to the curvature. In this case the lowest board is staked to

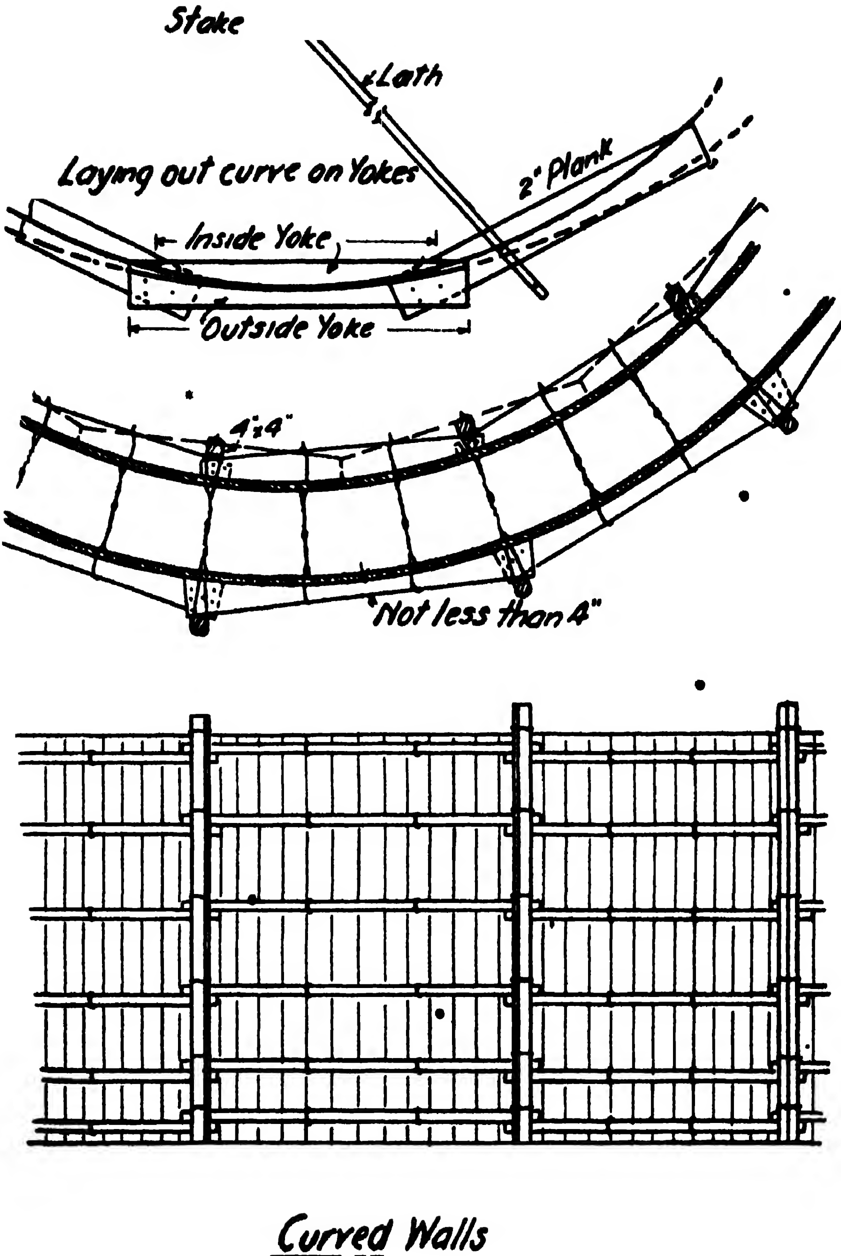

Stake
Lath
2" Plank
Laying out curve on Yokes
Inside Yoke
Outside Yoke
4"x4"
Not less than 4"
Curved Walls

FIG. 40.

line and the studs then nailed on. No wales can be used without wedging out from the studs, so there should be some outside bracing.

If a wall is a complete circle, as in tanks and silos, and often in small foundations, it is possible to dispense with the ties, but in this case the timber of the outside yokes is in tension and the nails or bolts at the lap of the yokes in shear. Since it takes about ten 20*d.* spikes to equal one ⅝-in. bolt in shear, bolts should be used to fasten the yokes together at least two to a joint If the wall is very low and of small diameter, five 20*d.* spikes can be used at each joint instead of the bolts. Concreting

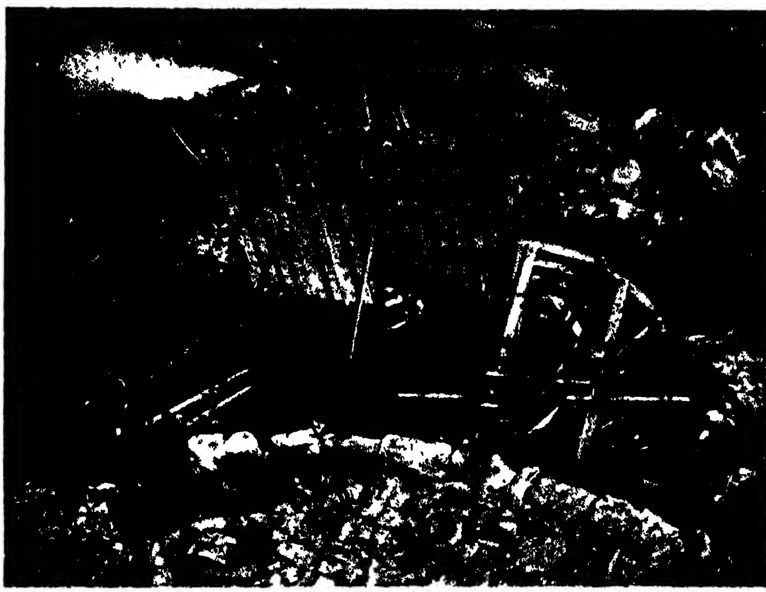


FIG. 41.—CIRCULAR WALL FORM SHOWING METHOD OF BRACING.

must be brought up equally around the wall, otherwise the forms will be thrown out of shape.

Batter-Curved Walls.—Curved wing walls for bridges, bridge piers, foundations for turbines, etc., are often battered. The yokes at different heights will have different radii, which must be calculated. The radius at the bottom will be greater than that at the top, so that the width of each board should really taper from bottom to top. This, however, is seldom done. General practice is to nail on three or four boards of the regular width, leave a space at the top of about 2 in., then nail on some more boards, leave a space, and so on. Each space is then filled with a special board cut to fit. It is best to use square-edged boards with this method so that the wedge pieces can be slipped in without having to cut off the tongues (*Fig.* 42).

Stripping.—After the exterior braces are removed the wires are cut or the clamps or nuts removed. There should be a box handy in

which to drop the parts, otherwise a large number will be lost. If the forms have been built in place, the wales are then knocked off and the studs pried away individually from the sheathing; the advantage of using as few nails as possible will be seen here As the studs have to be taken apart from the sheathing, it is easier to do it when they are standing in position rather than try to take sheathing and studs down together and then remove the studs; there is also the advantage of being able to pile the studs together without sorting them out from the sheathing The sheathing boards are then pried from the wall, and lastly the wires cut off or the bolts or rods pulled Sometimes the bolts are pulled before stripping, but generally the timber is required first for use elsewhere and so the bolts are left until last.

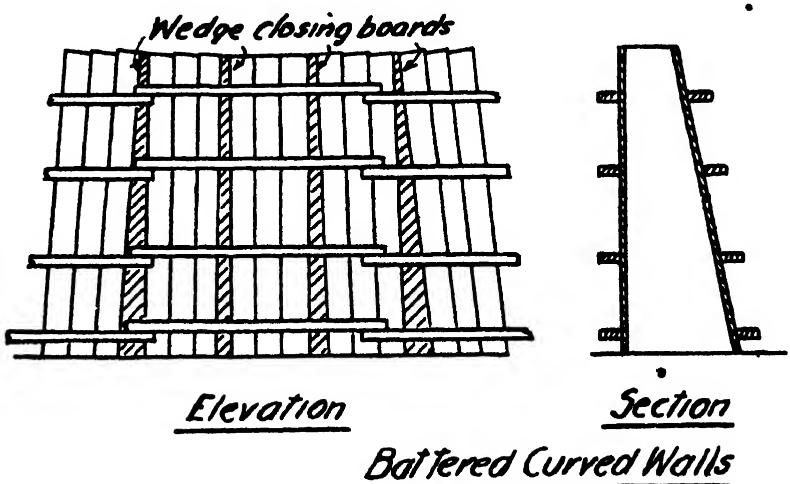


FIG. 42

In panel construction the panel is removed as a whole, prying out from the top of the wall In this case the bolts are generally removed first, as the panel can be stripped much more easily, but if not the wales are knocked off before removing the panels

In the summer forms can be removed in from 12 to 24 hours, but in cold weather they should be left 3 or 4 days, or until the concrete cannot be indented with the thumb nail The timber should be cleaned of concrete as soon as possible after stripping, as hardened concrete is difficult to remove Different sizes should always be piled separately, as much time can be lost in searching for studs under a pile of sheathing boards. Regular carpenters' stripping bars should be used for stripping, and when it is necessary to pry from concrete a short board should be slipped between the concrete and bar and care taken not to break off edges.

Estimating Cost.

The amount of timber required per square foot of contact area will increase with the height of the wall if the whole height is poured at

one time, and will depend on whether the timber sizes are the minimum required for strength or whether they are made heavier to allow for wear and tear.

It will vary from a minimum of 0·175 cu. ft. to a maximum of 0·35 cu. ft. per sq. ft. of contact area, measuring both sides of the wall. For ordinary building walls up to 10 or 12 ft. high an average of 0·225 cu. ft. per sq. ft. can be used. For high retaining walls of 20 ft. and over 0·30 cu. ft. is an average value, and proportionally for intermediate heights.

If the forms are built in panels and used several times these units will be divided by the number of times used, based on the whole area of both sides the wall. But for each time the panel is used 10 per cent. should be added to the amount allowed to make up the panels to cover possible repairs. That is, if the first erection takes 0·25 cu. ft. per sq. ft. and the panel is to be used four times the amount of timber to estimate is $\frac{0{\cdot}25 + 30 \text{ per cent}}{4} = 0{\cdot}08125$ cu. ft. per sq. ft. This limits the number of times the original timber can be used to ten. However, a panel is not usually used more than six times. These units will include waste in cutting, braces, spacers, etc.

The labour cost will also vary with the height, but is fairly constant for ordinary walls up to 12 ft. high. Above this height there will be an additional cost for raising the timbers, splicing the studs, and for the inconvenience of working above ground. For ordinary heights the labour cost per cubic foot of timber will be fairly constant, that is the cost per sq. ft. will increase with the amount of timber used per sq. ft., because there will be more weight to handle.

The thickness of the wall does not have much effect on the cost, though very narrow walls, such as 6 in. or 8 in., or very wide walls 3 ft. and over, will generally cost slightly more than the usual widths of 12 in. to 30 in., because in the narrow walls the carpenters cannot work inside the wall and in the very wide walls the ties will take a little longer to place.

Panel forms will cost more than forms built in place if only used once, but if used more than once will cost less.

For long retaining walls, where the forms can be used many times over, the cost of renting steel forms should be investigated.

Allowance for the cost of cleaning and oiling must be made each time a panel is used. A labourer should clean and oil 100 ft. super per hour.

When the forms are done with an additional cost for pulling all the nails must be added before the timber can be salvaged for use on another job. This is often overlooked, though it may amount to a considerable sum on a large job. A labourer should clean timber of nails at the rate of about 10 cu. ft. per hour and the cost should be figured on the amount of timber it is expected to salvage for use on the same job or on another job.

The cost of nails, wire, and bolts is usually allowed for by a lump

sum, rather than reckoning the cost for each item of formwork. The number of wire ties required will be approximately the total contact area divided by 1½. If bolts or rods are used this number should be divided by the number of times they can be used plus an allowance for waste and loss. About 1 lb. of nails per 10 ft. super of forms should be allowed, as the waste is large.

For walls up to 12 ft. high built in place it will require about 6 hours carpenters' time and 3 hours labourers' time to erect and brace 100 ft. super of forms, and 1 hour carpenters' time plus 2 hours labourers' time to strip them. To build 100 ft. super of panel forms will require about 3 hours carpenters' time plus 1½ hours labourers' time to make the panels, 3½ hours' carpenters' time plus 3½ hours labourers' time to erect them, and ½ hour carpenters' time plus 2 hours labourers' time to strip.

In either case the labourers' time cleaning must be reckoned each time the timber is used.

Cost of 100 ft. super of wall forms built in place—

7 hrs. carpenter @ .... =
6 ,, labourer @ .... =
Timber, say, $0{\cdot}225 \times 100$ = 22·5 cu. ft @ .... =

Cost of 100 ft. super of wall forms built in panels and used 4 times—

1st erection: 7 hrs carpenter @ .... =
8 ,, labourer @ .... =

Timber, 0·25 + 30 per cent. $= \dfrac{0{\cdot}325 \times 100}{4} = 8{\cdot}125$ cu. ft. @ .... =

For each of following erections:

4 hrs. carpenter @ .... =
6½ ,, labourer @ .... =

Or average cost for whole job =

4¾ hrs. carpenter @ .... =
6⅞ ,, labourer @ .... =
8⅛ cu. ft. @ .... =

CHAPTER IX.

DETAIL CONSTRUCTION OF BEAM AND GIRDER FLOOR FORMS.

THE method of calculating the size of timbers required for the several parts of a beam and girder floor has been shown in the previous chapters, so it will only be necessary now to describe the methods of framing and stripping. There is only one general method of framing, but there are several ways of constructing details to facilitate stripping

The method of constructing the details will determine whether the formwork is economical or expensive The main thought to bear in mind when designing the forms is the ease and sequence of stripping. A form is only temporary, and it must be built so that it can be taken apart with the least cost and the maximum salvage of material. The speed required on the job, and, closely allied to this, the need for reshoring the beams, will also affect the details of erection

It may be thought by some that reinforced concrete building is slow because of the time required for the concrete to set. This is not true, however; with proper attention paid to details of formwork and reshoring of beams a reinforced concrete building can be put up as fast, if not faster, than any other type of building It is common practice in the United States to complete a floor every week or ten days, whatever the size of the building may be.

In a multiple-story building it is customary to build only one complete set of forms for one floor, or for a floor and a half, and use these again on the upper floors, so that it is necessary to strip as soon as possible, and the forms must be designed so that the least important parts can be stripped first without disturbing those members which carry most of the load.

In beam and girder construction the unit method is always used, that is, the slab panels, beam and girder sides and bottoms are built up in advance to the correct size ready for erection, often going a step farther and building up the beam and girder boxes complete. A power saw is a necessity for economy, as there will be many small pieces of the same size to saw.

Timber sizes for the different parts will be practically the same for all jobs, except that the joist size may vary somewhat with the span. Before deciding on the joist to use, comparison for economy in material should be made between two or three different sizes. The standard sizes used for all ordinary buildings are as follows :—

For slab panels, 1 in. by 6 in. dressed to $\frac{7}{8}$ in. by $5\frac{5}{8}$ in. tongued and grooved.
,, joists, 2 in. by 6 in., dressed or rough.
,, beam ledgers, 1 in. or $1\frac{1}{2}$ by 4 in. or 6 in., dressed to $\frac{7}{8}$ in. or $1\frac{1}{4}$ in.

For cleats or battens, 1 in., $1\frac{1}{2}$ in. or 2 in. by 4 in. dressed.
,, beam and girder bottoms, 2 in plank dressed four sides.
,, ,, ,, ,, sides, 1 in. by 6 in. dressed if only used once and $1\frac{1}{2}$ in. by 6 in. dressed if used more than once.
,, posts, 3 in. or 4 in. by 4 in. dressed or rough.

In choosing between dressed and rough timber, faces coming in contact with the concrete should always be dressed, and it is customary to order material for slab sheathing, joists, beam sides and bottoms to be dressed four sides Beam bottoms may be dressed on one side only if thereby a single plank, or two planks together, will give just the required width of beam. Joists can be rough, but are harder to handle and the ends will have to be sized to an even depth. Posts are generally rough and are stronger than when dressed, but if they can be used elsewhere, as for studs, they should be dressed.

The bill of material should be made out from sketches of the forms, and not ordered haphazard Required lengths particularly should be noted, otherwise there will be an astonishing percentage of waste. The sketches for the different parts should be given to the carpenter foreman so that he will know for what part the different items were ordered.

If there is a basement wall to build, the timber can be used again in the floors and should be ordered with that in mind; that is, it may be cheaper to use 3 in. by 4 in. or 2 in. by 6 in. studs in the wall and re-use them as posts or joists in the floor rather than order 2 in by 4 in. studs and have no use for them afterwards.

As the various parts are framed up as units it will be convenient to consider them separately before describing the method of erection and joint details.

Slab Panels.—The average size of a slab panel between beams and girders will be about 5 ft. by 18 ft. This can be built in one, two, or four sections, the size depending mainly on the facilities for handling from floor to floor. With a hoisting engine on the job one panel would be the most economical, but if they are to be handled by hand two or four panels per bay would be better.

The panel will rest on the top of the beam and girder sides, and should be $\frac{1}{2}$ in. to 1 in. less in width and length than the clear dimensions between the concrete. They will thus set only partly on the sides, allowing $\frac{1}{4}$ in. to $\frac{1}{2}$ in. clearance all around for "give" in the top of the sides—and they will always give slightly under the pressure of the concrete—and so prevent the panel binding in the concrete and hence being difficult or impossible to strip. The edges all around should be bevelled to 45 degrees.

The boards are nailed together with 1 in. by 4 in. battens spaced the same distance apart as the joists will be, and so that they will be about midway between joists. The battens must be about 2 in. shorter than the width of the panel. As panels are liable to swell if left long exposed to rain, to prevent buckling an end board is often left off until the last minute; or if two panels are placed side by side a narrow strip is left between them afterwards filled in or covered with a strip of tin or heavy paper.

Another method of construction is to nail the boards direct to the joists, using them as battens. This will save material and labour but it makes the panels heavier to handle, so should only be used when there is sufficient equipment to handle them. The panels are made in one section for each bay. Each end board is nailed on lightly for easy stripping, since if they stick they will remain in place while the rest of the panel can be removed. Sometimes the length of the panel is made a board-width short each end and these boards nailed on lightly to a joist placed a board-width away from the girder, for the same purpose of easy stripping. One of the end boards is left off until last to allow for swelling and to provide an opening for cleaning out the forms.

If the panels are to be used only once, unless it is desired to make them up in advance, loose boards can be used without the battens.

Joists or Spreaders.—The length of the joists will be the clear distance between beams less twice the beam side thickness less ½ in. The ends should be bevelled inwards at the bottom about ¼ in. so that they can be easily stripped without binding If rough joists are used the ends must be sized to an even depth to give a level slab. When they are nailed direct to the sheathing the length and bevel will be the same.

Beam and Girder Bottoms.—The width of the plank should preferably be the same as that of the beam so that cleats will be unnecessary, but, as the cost increases with the width, above about 10 in. wide it is cheaper to use two planks nailed together with 1 in. by 4 in. cleats on the underside, 24 in. to 30 in. apart. The cleats must not be longer than the width of the bottom.

The length of the bottom will depend on whether it is to be carried on the column sheathing or blocked up from the yokes and butted against the sheathing. In the former case the length will be the clear span between columns less ¼ in. to ½ in. clearance at each end, and the ends will be bevelled inwards at the top. In the latter case the ends will be square, and the length will be the clear span between columns less twice the column sheathing thickness. In the first case a little leeway in measurement is allowed, but in the latter the measurement of the bottom must be accurate. The former method is generally used when the bottoms are to remain in place after the sides are stripped and the latter when the sides and bottom are stripped together as a unit, as it gives a little more clearance after the columns are stripped. When the bottom is carried on the sheathing the column is a little easier to strip. When the beam bottoms are carried on the girders either

construction may be used and the length will be accordingly, the girder taking the place of the column.

Girder Sides.—The depth of the side will be measured from the underside of the slab sheathing to the bottom of the girder bottom. The side is usually made exactly this depth, but to save ripping a board all full-width boards may be used and the side allowed to project below the bottom, in which case notches will have to be cut at the posts to allow the bottoms to bear on the caps. The sides should never bear on the bottom, but always overlap. The length of the side will depend on the system of framing used.

The best method is to make the length of the side equal to the clear span between columns less twice the thickness of the column sheathing less about 1½ in. each end for clearance, that is, if the columns are 18 in. square and 18 ft on centre and the sheathing used is 1¼ in. the girder side will be 16 ft 0½ in long. The ends are bevelled outwards and the 1½ in. clearance is afterwards filled in with a bevelled batten, as mentioned later This method is used with a column head as shown at (*b*) *Fig.* 12.

Another method is to make the length equal to the clear span less twice the sheathing thickness, or the ends of the sides will be flush with the column sheathing and are covered with an independent bevel strip. Still another method is to carry the sides over the column sheathing but stopping ¼ in. to ½ in from the inside face and bevelling the ends inwards; in this case the length will be the clear span less ½ in. to 1 in.

The latter two methods, used with column heads as shown at (*c*), *Fig.* 12, make the column sides less easy to strip and leave small fins on the concrete at the junction of side and bottom bevels unless the space is filled in. The first method will be considered best practice, and the slightly extra labour cost will be more than saved when it comes to stripping.

The boards are nailed together with battens 24 in. to 30 in on centre, the size depending on the depth, using 1 in. by 4 in battens up to 15 in., 1½ in. by 4 in. from 15 in. to 24 in.; 2 in by 4 in. from 24 in. to 36 in.; and 3 in. by 4 in. for greater depths. The boards should be well-clamped together before nailing, to close up the joints. The end battens should be a few inches in from the ends.

The openings for the beams correspond to the openings in column heads for the girders, and can be made by either of the methods shown in Chapter VII, and in *Fig.* 43. The former is the better; the width will be ½ in. to 1 in. wider than the width of the beam, the depth will be ¼ in. to ½ in. greater than the depth of the beam, and all edges will be bevelled inwards. If the beam bottom is carried on the sheathing, as must be done anyway if there is not much difference in depth between the beam and girder, the opening must be the depth of the beam plus beam bottom and the bottom edge will be left square; and the width of the opening will be that of the beam plus twice the beam side thickness. To support the bottom a short piece of 2 in. by 2 in. or 4 in. is nailed

under the opening in either case. Temporary cleats should be nailed across the openings until the sides are erected and braced.

If the beams are deep, the openings will leave the girder sides weak, and they will be liable to break when stripping. A good method of overcoming this is to build the sides up to the underside of the beam bottoms as a unit with the girder bottom, nailing them permanently together and stripping together. The remaining portion of the sides is then built as separate units from column to beam and beam to beam, leaving 1½ in. clearances at each end if desired, and letting the battens project below the sheathing for lap (*Fig* 44).

Beam Sides. These are built the same way as girder sides, preferably leaving 1½ in clearance each end as mentioned. The connection to the girder is the same as the connection of the girder to the column

Beam Ledgers. These are nailed on to the side battens at a distance down equal to the depth of the joists They should stop off a few inches from the ends of the sides. The top surface must be level so that the joists will not have to be wedged up or cut down to give a level floor For spans up to 6 ft, 1 in by 4 in will be sufficient; for greater spans 1½ in. material should be used Instead of a continuous ledger, 6 in. by 6 in blocks by 1½ in. thick are often nailed on the beam sides at the spacing of the joists This will save some material, as waste boards can be used and the joists will be easier to strip

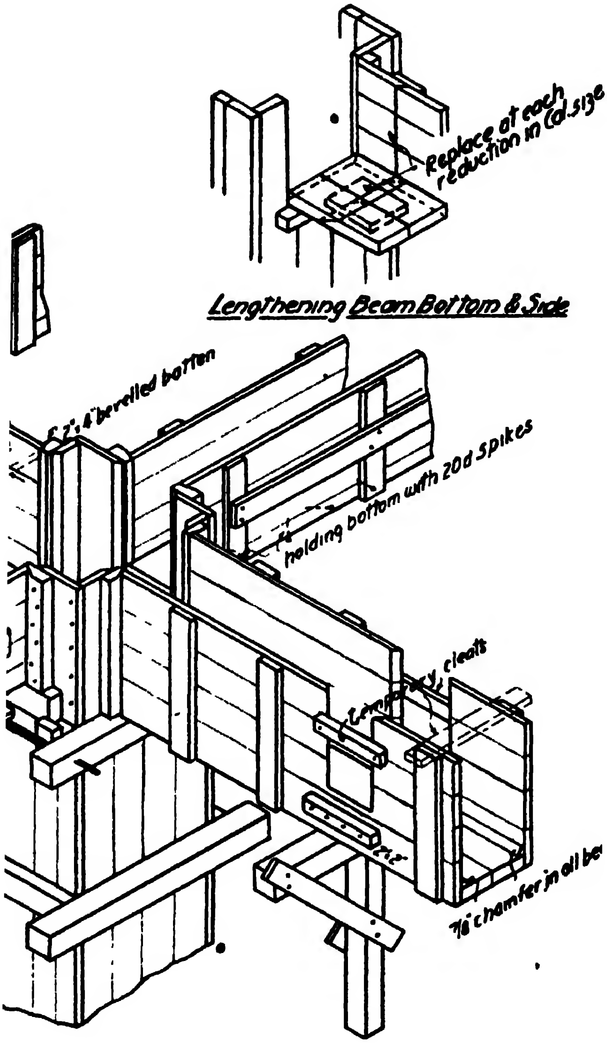
Shores To the top of the post is spiked a cap of the same size as the post and about 15 in to 18 in longer than the width of the beam. The ends of the cap are diagonally braced back to the post by short pieces of 1 in by 4 in or 6 in on opposite sides The posts should be carefully cut off square at each end to obtain full bearing The posts will rest on wedges, which should be hardwood, 8 in to 10 in. long, 4 in to 6 in wide, and tapering from 3 in down to about ½ in. The wedges are generally placed on a 2 in by 8 in. sill about 12 in. long. If bearing on soft ground the size of the sill should be calculated to give a low pressure on the soil, not exceeding 1 ton per sq. ft The required length of post should be noted before ordering, as this is a place where there can be much waste. Spliced posts should be avoided, but they are sometimes necessary, especially when a story height changes. They should have long cleats at the joint on all four sides, well nailed, and the abutting ends of the posts should be square

The posts if longer than 8 ft. will have to be braced at the centre with 1 in. by 6 in. boards in both directions, placed horizontally.

Erection.—*Fig* 43 shows the assembled forms and the different methods of building the units and making the connections. The different parts being ready to erect, the carpenters should work in three gangs, one placing bottoms, another sides, and the last one joists and slab panels.

The outside wall columns are first set and plumbed, and on these are placed the beam bottoms all around the building, two corner columns

FIG. 43



Lower Girder shows connections when beam bottom and sides over lap column or girder sides.

Upper Girder shows connections when beam bottom and sides do not over lap column or girder sides

r Beam and Girder Floors

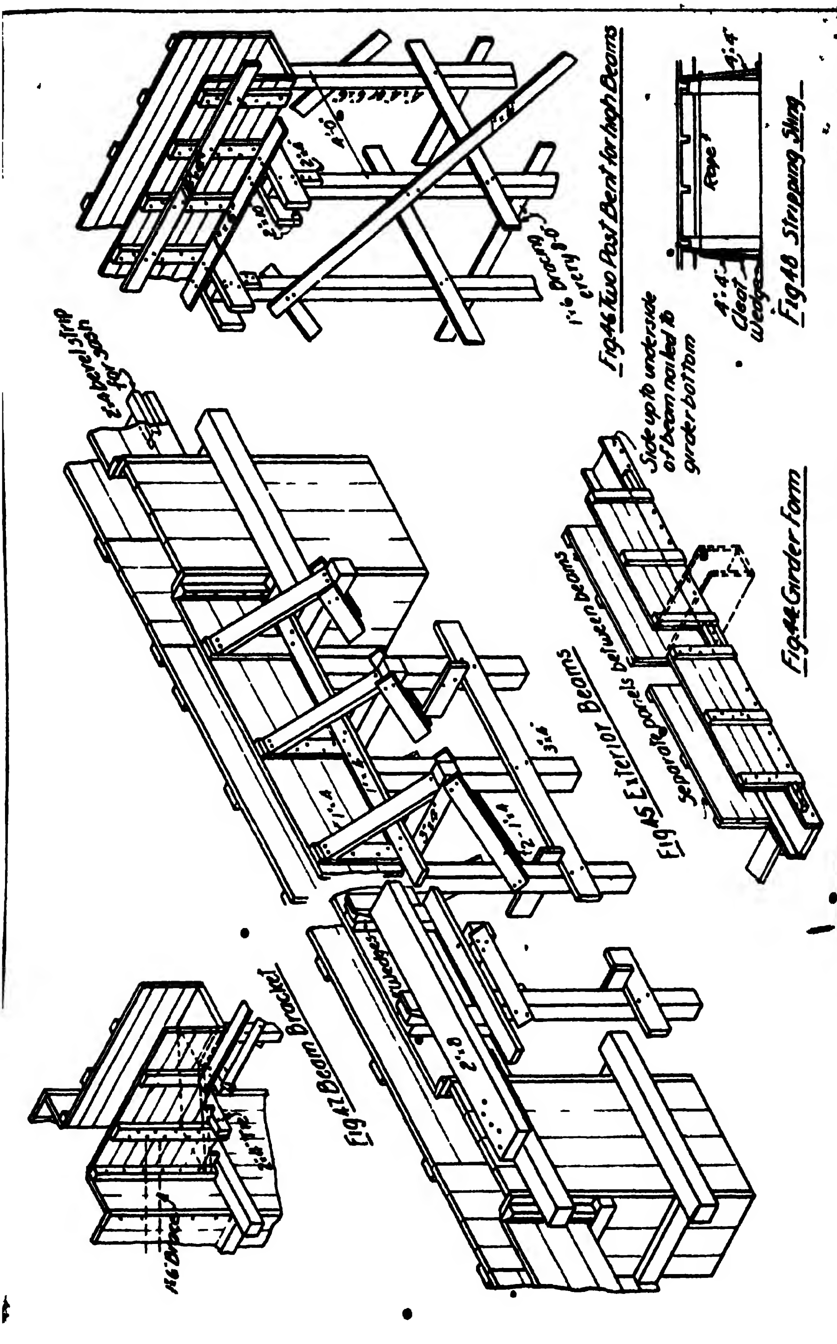


Fig 46 Two Post Bent for high Beams
1'-6" Bracing every 8'-0"
Fig 48 Stripping Sling
Rope
4"x4" Cleat Wedges
Side up to underside of beam nailed to girder bottom
Fig 44 Girder Form
2x4 bevel strip for wash
Fig 45 Exterior Beams
Separate panels between beams
Fig 47 Beam Bracket
Wedges

being set first to give the line. This establishes definite points to work from, and the carpenters work in towards the centre of the building, setting next the first interior row of columns and placing the girder bottoms from these to the outside columns, then taking the next row, and so on.

One 20*d.* spike is placed in each end of the bottom and not driven home until the spacing of the columns has been checked with the tape. A shore is put under the centre and a carpenter walks along the plank and drives a spike into it through the bottom, and then the remaining shores are placed and spiked The sills and wedges are inserted under the posts—they should not be placed at the top—and the wedges are driven tight so that the bottom has a camber at the centre of about ¼ in in 10 ft.; this is particularly necessary when shoring from the ground, as there will always be some settlement.

If the girder bottom is not carried on the column sheathing, the top yoke should be placed at the right height to carry the bottom, and beam bottom is blocked up from this yoke.

The girder sides are lifted into place and lightly nailed to the bottom to hold them in place, and braced across the top to prevent spreading. The beam bottoms between girders are next placed and shored up It is important that all shores are placed plumb to avoid eccentric loading, and hence bending in the shore, this is particularly true of spliced shores. The beam sides are then placed, lightly nailed and braced at the top; they should be set level. If 1½ in. clearance has been left, 2 in. by 4 in. battens bevelled on one side are now inserted and nailed to the column or girder. They are put on now instead of when making the sides so that the bevel can be cut to suit the exact opening. The centre of the girder sides should be braced back to the beam sides

If the joists are not nailed to the floor panels they are now laid on the ledgers at the required spacing, leaving them loose. When they are levelled up the floor panels are dropped into place; these also are not nailed down, except for one or two nails at the corners to hold them in place. The panels must be notched out around the columns; this is more easily done in place than beforehand. The lower edges of the beams and girders are usually covered with a ⅞ in chamfer strip nailed to the bottoms.

The bottom of the sides must be held against the pressure of the concrete, and this is done in various ways. Beams up to 24 in. total depth can be held by driving in a 20*d.* spike at each batten and two intermediate ones driven into the bottom on the slant and in opposite directions, leaving the heads projecting for easy stripping.

Without spiking the sides at all the best method is to nail a 1 in. by 4 in. or 6 in. ribbon on top of the shores on each side of the beam, and this will hold any ordinary beam. Sometimes the side battens are carried below the beam and tied across with short pieces of 2 in. by 4 in. below the bottom. Patent clamps are also used, but they are expensive, easily lost, and sometimes a carpenter forgets to tighten

one up. Bolts above or below the bottom are also used, particularly when heavy battens are required; and it may be necessary to use a bolt or wires half-way up the beam if it is very deep, as then the side becomes similar to a wall form, even perhaps requiring a horizontal wale.

The last operation is to cross-brace the shores. Before commencing to pour, the wedges should be gone over and tightened up where necessary, and it is advisable to have a carpenter under the floor watching the forms, particularly the posts and wedges, all the time during concreting. Instead of erecting beam sides and bottoms separately, the beam boxes can be built as a unit; that is, sides and bottoms are nailed on the ground and erected and stripped together. One end of the box is up-ended to its position on the column, and the carpenters take the other end on their shoulders and with the aid of a post raise it up on to the top yoke of the other column There will be more weight to handle, though fewer pieces, and the fitting can be done more accurately. The sides are spiked to the bottom as explained before, and ribbons are not used unless the beam is over 24 in. deep. The sides and bottom need not necessarily be stripped together, though they should be to use this method to the best advantage. It is generally economical for a one-story building, since the beams are light and the forms can be left in until they can all be stripped at the same time. In a multiple-story building, however, speed will generally require the sides to be stripped before the bottoms, so they must be independent units and half the advantage of this method is lost.

Nailing the joists to the floor panels should be done where it is possible to handle the panels, because there will only be one unit to place instead of several. Isolated beams with no slabs must be held at the top as well as at the bottom, either by wood ties, bolts, or wires. It will be noticed that few nails are required during erection; where they have to be withdrawn double-headed nails are an advantage All these details are shown in *Fig.* 43.

Exterior Beams (*Fig.* 45).—Exterior beams along the building line and interior beams around a stair opening or elevator will have no slab on one side to brace the top, so some other means must be used. The commonest method is to make the post caps longer on the outside and run 1 in. by 4 in. diagonal braces from the caps to the top of the sides, preferably bearing against blocks to prevent slipping. It is a little more difficult to align the side with this than with the following method, and also more posts are required.

Instead, a 2 in. plank (scaffold plank will do) can be spiked to the top column yokes of adjoining columns and the sides held to line by driving wedges between the plank and side. It is then easy to rectify the alignment if required by tightening or loosening the wedges. The bottom of the side is held as before with a ribbon or spikes. Posts should be placed at least as close as with interior beams, although the load is less; and they should have firm bearing, since settlement will look bad and interfere with the setting of the sash. No camber should be given

the beam if a sash frames into it. A bevelled 2 in. by 4 in. must be placed in the bottom to form a slot for the sash or as a nailing strip for a wood frame.

Stripping.—The length of time the forms must be left in place before stripping depends on so many conditions, often peculiar to a particular job, that no definite rules can be given, and it is a question that should be decided by the engineer or architect. For an ordinary job, with no special features, the temperature of the air is the most important point. The setting qualities of the cement, the span of the member, and whether the member is under compression only or is subject to bending, are also important points. Long spans should be left longer than short spans, and beams and slabs longer than columns and walls. As a guide the following is given, but it must be combined with judgment. Summer temperature will be taken as 60 deg Fahr, and winter temperature as 40 deg Fahr; for intermediate temperatures the time can be taken accordingly

Girder and beam sides can be stripped in		3 4	days	with temp	at	60	deg. F.
	,,	9-10	,,	,,	,,	40	,,
Slabs up to 6 ft. span ,, ,,	,,	5 6	,,	,,	,,	60	,,
	,,	12	,,	,,	,,	40	,,
Beam and girder bottoms ,,	,,	8 10	,,	,,	,,	60	,,
and long-span slabs ,,	,,	21	,,	,,	,,	40	,,

These times are conservative Below 40 deg it is mainly a question of the heat and protection given during and after pouring.

All beams must be reshored immediately until the concrete has been poured for at least 28 days, as they must support the construction on the upper floors as well as their own weight.

The order of stripping after the column forms are removed depends somewhat on the method of framing and whether the bottoms are to remain after stripping the slab If the bottoms and shores are to remain in place the order of stripping is as follows: ribbons holding beam sides; bevelled keys or cleats at junction of beam and girder, ledgers; joists; girder sides; beam sides; floor panels; beam bottoms and shores; girder bottoms and shores. This means that all the forms except the bottoms are stripped together. If the sides are required first and the slab panels left in place a little longer, each shore in turn is loosened, turned around with the cap parallel to the beam, and wedged up again, then the girder sides first and the beam sides next can be taken down, leaving the panels in place and putting temporary shores under them.

If the slab panels have been built with loose or lightly-nailed boards all around the outside they can be removed with the joists before stripping the sides.

If loose joists with a continuous ledger are used the ledger must be taken off first, but if the joists are loose and carried on blocks they can be knocked off without removing the blocks.

If the panels are nailed to the joists the ledgers or blocks must be removed first unless the shores are turned around and the beam sides removed.

If the bottom and sides are built as a unit the order of stripping will then be: ribbons; cleats and keys; shores under girders; girder sides and bottom together; shores under beams; beam bottom and sides together; and lastly the slab panels.

All beams and girders must be immediately reshored and temporary shores should be placed under the slab panels to prevent them falling while the beam boxes are being removed.

When girder sides are built as in *Fig.* 44, usually the lower half of the sides and the bottom are stripped first, then the girder sides, beam sides, and slab. Small pieces, like battens and cleats, which have to be removed should be tacked on the form to prevent loss

When the bottoms are left in, the shores and bottom are stripped together and not the shores separately Shore braces are removed before beginning to strip but are replaced on the permanent shores.

A light scaffold must be built for removing the cleats, ledgers, ribbons, etc. The beam sides and slab panels after being pried loose with stripping bars are allowed to fall on to two rope slings and eased to the floor to prevent breakage To make these slings, two 4 in. by 4 in. posts are bored through near the top to take a rope. The rope is knotted at one post and is carried down and fastened around a cleat on the other post. The rope should be long enough to cover one bay between columns. The posts are wedged up tightly between floors alongside the girders, and they should be a little longer than the clear story height. Two slings per bay will be sufficient (*Fig.* 48.)

Panels should be immediately cleaned of any adhering concrete.

Re-erection.—The size of the columns will usually reduce on the upper floors so that when re-erecting the forms it will be found that the beam and girder forms framing into the columns will be too short because of the longer clear span The girder bottom is lengthened at each end by half the increase in span by splicing on pieces of plank with cleats; the joint will be stronger if bevelled, and it must be done each end to keep the beam spacing symmetrical. The sides are similarly lengthened by cleating on a narrow strip at each end. For each reduction in size of column it is better to take off these strips and add wider ones so that there will be only one patch instead of several. On upper floors these patch strips can be used to give the necessary clearance for stripping instead of using the bevelled battens, as they can be taken off before stripping the sides.

The slab panel will also have to be pieced out at the columns. This can be done as for columns by cleating on short pieces, but it makes rather a clumsy joint as the edges of the panel are bevelled where it rests on the column, so the new pieces have also to be bevelled to fit or else the bevels are sawn off. A neater joint is made by sawing off each corner

of the panel where it comes against the column on the diagonal, so that it will just miss the corner of the column, at the time the panel is made up. The corner is then filled in with one or more boards parallel to the diagonal, cleating on the outside board. At each reduction these boards are removed and new ones put in their place.

If beams reduce in depth on upper floors the sides are simply dropped the required amount (not sawn off at the bottom) and notches are cut for the post caps. If the depth increases it will be an advantage to have had the sides long in the first place, so that only the notches will have to be patched and not the whole form.

When building the forms in the first place the required changes on upper floors should be carefully noted and the forms designed so that the changes can be made with the least expense.

Re-shoring.—In order to maintain a satisfactory speed it will generally be necessary to pour a floor when the floor below has not attained sufficient strength to support the load coming on the shores, so that after stripping the lower floor some or all of the shores must be replaced. These should be put back immediately after stripping and wedged up again under the beam, but not so tight as to strain the concrete. It is best to order sufficient beam bottoms for two full floors and sufficient shores for two and a half floors, so that the second floor can be fully formed and poured before removing the first floor shores—the sides and slab panels can be used on the second floor. The first floor is then stripped and half or two-thirds of the shores replaced; all the bottoms and half the shores are used on the third floor, and so on. In this way there is always one story fully shored and one story half or two-thirds shored below the floor being poured; in addition there should be a few shores in the third story below, say, one at the centre of each beam and girder, which are removed after concreting (*Fig.* 49). With re-shoring a floor can be poured every 8 to 10 days.

With long spans and heavy beams it may be necessary to leave the shores in for a considerable time, in which case to be able to use the timber in the bottoms, at the same time never leaving the beam unsupported, a special design can be made.

Spans of 30 ft. to 35 ft. should have one shore undisturbed during stripping, which can be done as follows (*Fig.* 50): Two saw cuts about 10 in. apart are made right through the beam bottom at the centre, giving a loose piece which is cleated on again during erection. A 6 in. by 6 in. shore is placed in the centre of the loose piece, only instead of the cap being placed on top of the post a short piece of 2 in. plank is placed each side of the post flush with the top, the nails projecting for easy withdrawal. The saw cuts are made at an angle of 45 deg. The remaining shores are placed in the ordinary way. To strip, the two cross pieces and all the shores except the centre one are removed, when the beam box can be lowered and turned around as a whole, or the sides can be stripped first and then the shores and bottom removed.

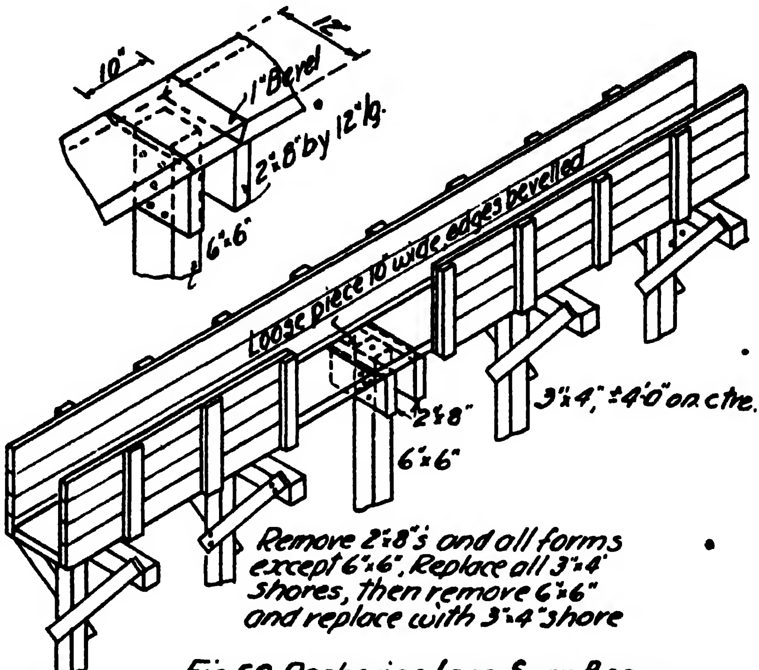


Fig 50. Reshoring Long Span Beams

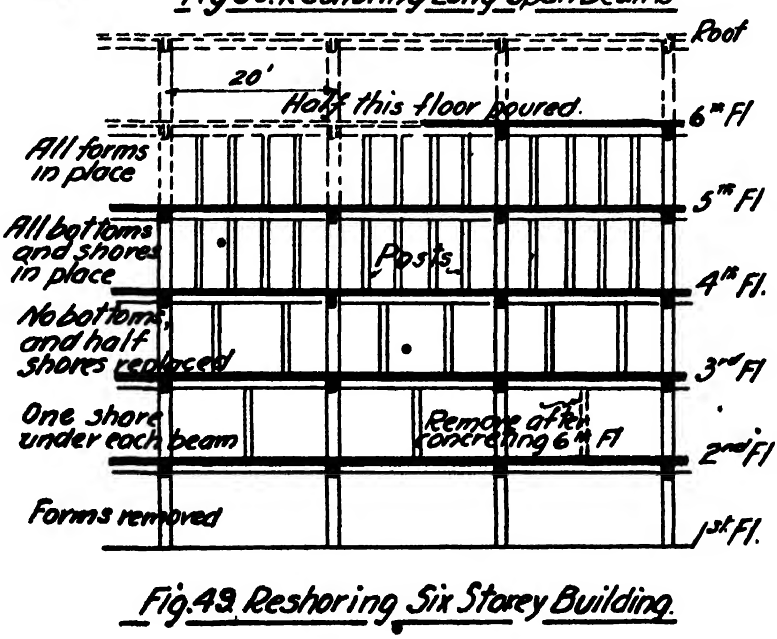


Fig. 49. Reshoring Six Storey Building.

All the shores are then replaced and the 6 in. by 6 in. can be removed if desired and replaced by a smaller post.

For longer spans still, two permanent shores should be left in the same manner on two loose pieces, and as the sides will be heavy they should be cut in two or three places and cleated together again after erection, so that each part can be removed separately by removing the cleats.

Replaced shores must be cross braced as when first used.

Very High Beams.—When the story height is exceptionally high, say 25 ft. and over, and the beams are heavy, it is inadvisable to use single post shores unless they are made heavy, two post bents are better. These are shown in *Fig.* 46, and consist of two 4 in by 4 in., or if necessary 6 in by 6 in., posts about 4 ft. apart, well cross-braced and supporting the beam bottom on two 2 in planks, one nailed on each side of the posts They should be deeper than required to carry the load so as to decrease deflection which would cause bending in the posts. Sway bracing should connect the bents longitudinally. If the slab span between the beams is long, ordinarily requiring a centre ledger, it would be better in this case to use a heavier joist sufficient to span from beam to beam and so avoid high shoring under the slab

Brackets.—If the brackets are not large they can be built as an integral part of the beam form, as in *Fig.* 47 There should be a shore at the start of the bracket, and the joint in the bottom should be on a bevel. If the brackets are large they will be clumsy to handle with the beam form, and are better made as separate units, cutting through the sides and cleating after erection. They can also be built as shown for column brackets.

Very heavy and long brackets should have the shores placed at right angles to the bottom of the bracket, and they must be held from slipping on the floor.

Estimating Cost.

Slabs and beams are usually estimated separately, as the proportion of slab area to beam area may vary considerably. The slab is measured over the whole area, not deducting for beams and girders, since the excess area—about 15 per cent.—will cover the cost of making bevels, fitting around columns, etc. Beams and girders are measured by the surface area in contact below the slab; that is, the area of two sides plus bottom, the length being taken centre to centre of columns or girders.

Slab forms, including joists, will require about $\frac{1}{6}$ cu. ft. of timber per sq. ft. of area. Beam and girder forms, including posts and bracing, will require about $\frac{1}{3}$ cu ft. of timber per sq. ft. of area in contact. The proportion of beam and girder area to slab area on an average job is about 50 per cent.—rather more than less—and since they take twice as much timber per sq. ft. a quick estimate of the amount of timber required can be obtained by figuring $\frac{1}{3}$ cu. ft. over the area of slab and dividing by the number of times the forms are to be used.

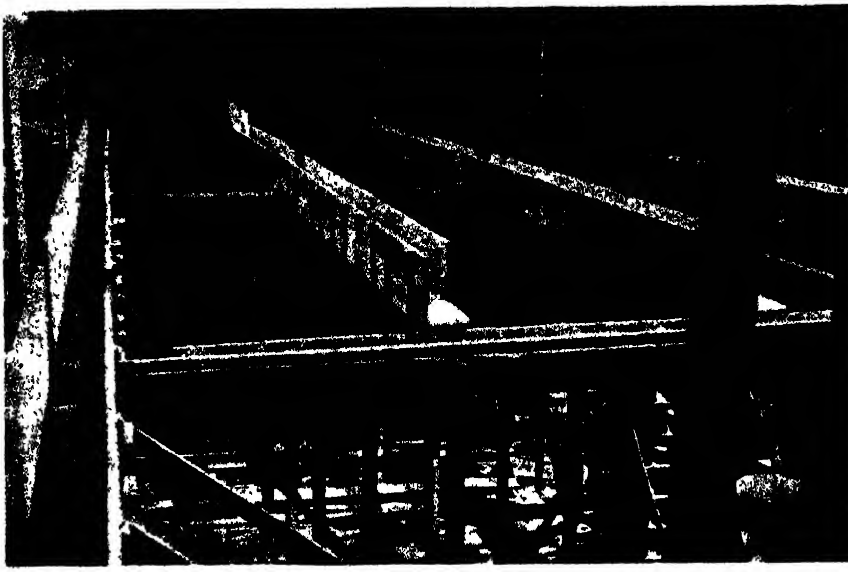


FIG. 51.—GIRDER SIDES BUILT TO UNDERSIDE BEAM BOTTOMS AND COMPLETED WITH PANELS FROM BEAM TO BEAM.

FIG. 52.—SUPPORTING EXTERIOR BEAMS DIAGONALLY FROM SHORES.

For re-shoring, a complete set of shores and braces will require about 0·15 cu. ft. of timber per sq. ft. of area of beam, or the number of shores required can be estimated at so much each.

About 10 per cent. should be added to the timber quantities for each time the forms are reused, to allow for repairs, etc.; this is a liberal allowance. Brackets are figured in with beams.

To make up 100 sq. ft. of slab forms, including joists, will require about 2½ carpenters' time plus 1 hour labourers' time. Erection in place will require about 1½ hours carpenters' time plus 1½ hours labourers' time, and stripping about ½ hour carpenters' time plus 2 hours labourers' time.

No difference is made whether the joists are loose or attached to the panels, in the former case assembly will cost less and erection more, but the total cost will be about the same, though fixed joists should work out a little cheaper.

Cleaning and hoisting will each require about 1 hour labourers' time.

Cost of 100 sq. ft. of slab forms, first erection :—

4½ hrs. carpenter at..........
4½ ,, labourer at ....
timber 100/6, say 17 cu ft at......

For each subsequent erection cost will be - -

2 hrs. carpenter at..........
5½ ,, labourer at ....
timber 10 per cent. of 17 = 1·7 cu. ft. at......

Cost of 100 sq ft of slab forms, used 4 times, calculated on whole area:

2⅜ hrs carpenter at........
5½ ,, labourer at..........
timber $\frac{17 + 5 \cdot 1}{4}$ say 5½ cu ft at......

Labour on beams and girders is figured at an average cost, although girders will actually cost a little more than beams.

To make up 100 sq. ft. of beam and girder forms will require about 4 hours carpenters' time plus 2 hours labourers' time. Erection will require about 5 hours carpenters' time and 5 hours labourers' time, and stripping about 1½ hours carpenters' time plus 3 hours labourers' time. Cleaning and hoisting will each require about 1 hour labourers' time.

Cost of 100 sq. ft. of beam and girder forms, first erection :—

10½ hrs. carpenter at.........=
10 ,, labourer at..........=
timber 100/3 = 33⅓ cu. ft. at......=

Each subsequent erection will cost :—

6½ hrs. carpenter at..........=
10 ,, labourer at...........=
timber 10 per cent. of 33⅓, say 3⅓ cu. ft. at...... =

Cost of 100 sq. ft. of beam and girder forms used 4 times :—

7½ hrs. carpenter at.......... =
10½ ,, labourer at...........=
timber 10·83 cu. ft. at........=

Both for slabs and beams 1 hour labourers' time is added for lowering the timber from the fourth floor to the ground, but no allowance is made for cleaning it again as some of it will be worthless, some may be sold as it is, or the cleaning may be charged to the next job.

For replacing and stripping shores about ¼ hour carpenter and ¾ hour labourer should be allowed additional for each shore. Additional labour should be allowed for brackets, from 1 to 4 hours carpenters' time apiece, depending on the size.

Bevel strips on beam edges should be estimated on the total number of lineal feet required. A carpenter will place 50-60 lineal feet per hr.

These costs are for ordinary conditions; extra high beams and unusual features will need additional labour charges.

CHAPTER X.

FORMS FOR RIB FLOORS AND STRUCTURAL STEEL FIREPROOFING.

Rib Floors.

THIS type of construction is commonly used when the live load is light and a flat ceiling unobstructed by beams is desired. The span of the slab may vary from 8 to 30 ft, but is generally about 20 ft. The ribs are formed by fillers, which may be made of hollow clay tile, steel, or gypsum, and occasionally of wood.

The filler is generally left in, although metal tiles are made which can be stripped and used again, wood forms are never left in. Whether it is economical or not to remove the fillers depends on the first cost and the number of times they can be used. Removable tiles are much heavier and hence cost more, and when the cost of stripping is added it is usually cheaper to leave the tiles in place unless they can be used many times over.

With metal tiles, unless the plastered ceiling is suspended, it is usual to lay the plaster lath direct on the forms, in which case the depth of the lath must be allowed for in calculating the height of the forms. When no plastered ceiling is required the tiles are removed, and in this case wood forms are sometimes built.

Hollow clay tiles are 10 in. to 12 in. wide and the ribs usually 4 in. wide, so that the tiles are placed 14 in. to 16 in. on centre. Metal and gypsum tiles are about 20 in. wide at the base, and the ribs 4 in. to 5 in. wide, so they are placed 24 in. to 25 in. on centre. Wooden fillers can be made any desired width and height, but are usually not over 30 in. wide and 12 in. high.

The spacing of the ribs mainly governs the type of form used. With clay tile fillers a " closed deck " is nearly always used; that is, the floor is completely sheathed over. For other fillers an " open deck " is used, that is, the sheathing is only under the ribs. With ribs 16 in. on centre if an open deck is used with 2 in. by 8 in.'s under the ribs the same amount of timber is required as with a closed deck. The 2 in. timber will cost considerably more than the 1 in., and will cost more to place, as the planks have to be spaced carefully and fillers will be required to form the T to the concrete beam. Also, with an open deck there is some possibility of losing concrete at the bottom of the ribs unless tile spacers are used. Instead of 2 in. by 8 in. plank, 2 in. by 6 in. may be

used, but this gives only 1 in. bearing for the tile, and the planks have to be spaced very accurately. There is, however, more salvage value with 2 in. timber than with 1 in., and in some cases, as, for instance, if it is known that the planks can be used again several times, it may be advisable to use an open deck with clay tile.

With ribs 24 in. on centre an open deck will save one-third of the sheathing, although the plank will cost more per cubic foot. The load in this case is concentrated at the ribs, so that is where the support should be; a closed deck would not distribute the load over 24 in.

Closed Deck. (*Fig.* 53.) This will be very similar to forms for beam and girder construction. Ledgers will be required to carry the joists; they may be single or double timbers, and should be cleated to the posts. If double they should be nailed together, and both timbers should not be spliced over the same post.

The floor sheathing can be loose boards or panels, depending on the number of times they can be used. The edges resting on the beam sides should be bevelled and have ⅛ in. clearance. The joists should be lightly nailed to the ledger here and there to keep them in place. The posts will be on wedges as before. The post cleats are preferably nailed to the posts before erection, so saving labour in erection.

Beam sides and connections to columns will be the same as for beam and girder construction, but since the depth below the slab is generally small a single board is often deep enough, so that no battens are required.

Beam forms are erected first, then posts and ledgers either separately or together. They must be set plumb and braced in both directions. To set to correct height a straight-edge can be used running on the beam side ledgers. Then the joists and sheathing are placed and the wedges are gone over to bring the floor level.

To strip, the beam sides are removed first as a unit and the beam reshored. The wedges under the posts are loosened and the posts and ledgers are removed together, putting in a temporary shore under the joists or leaving in a centre ledger until the joists are removed. If double ledgers are used, the nails at the splices—which should not be driven home—should be drawn before stripping.

If the joists are in short lengths, after loosening the wedges they can be knocked over and removed with the sheathing without removing the ledgers, and these can be wedged up again and left in longer. The slab forms should be left in as long as beam bottoms, and when stripped one or two lines of posts about 10 ft. apart should be replaced with the attached ledgers until the concrete has been poured for at least 28 days.

Since the width of the sheathing between beams will be much greater than in beam and girder construction, boards should be left out here and there to allow for swelling, or if panels are used there should be clearance between each panel.

Open Deck. (*Fig.* 54.)—Two-inch planks take the place of 1-in. sheathing, in other respects the construction is much the same for

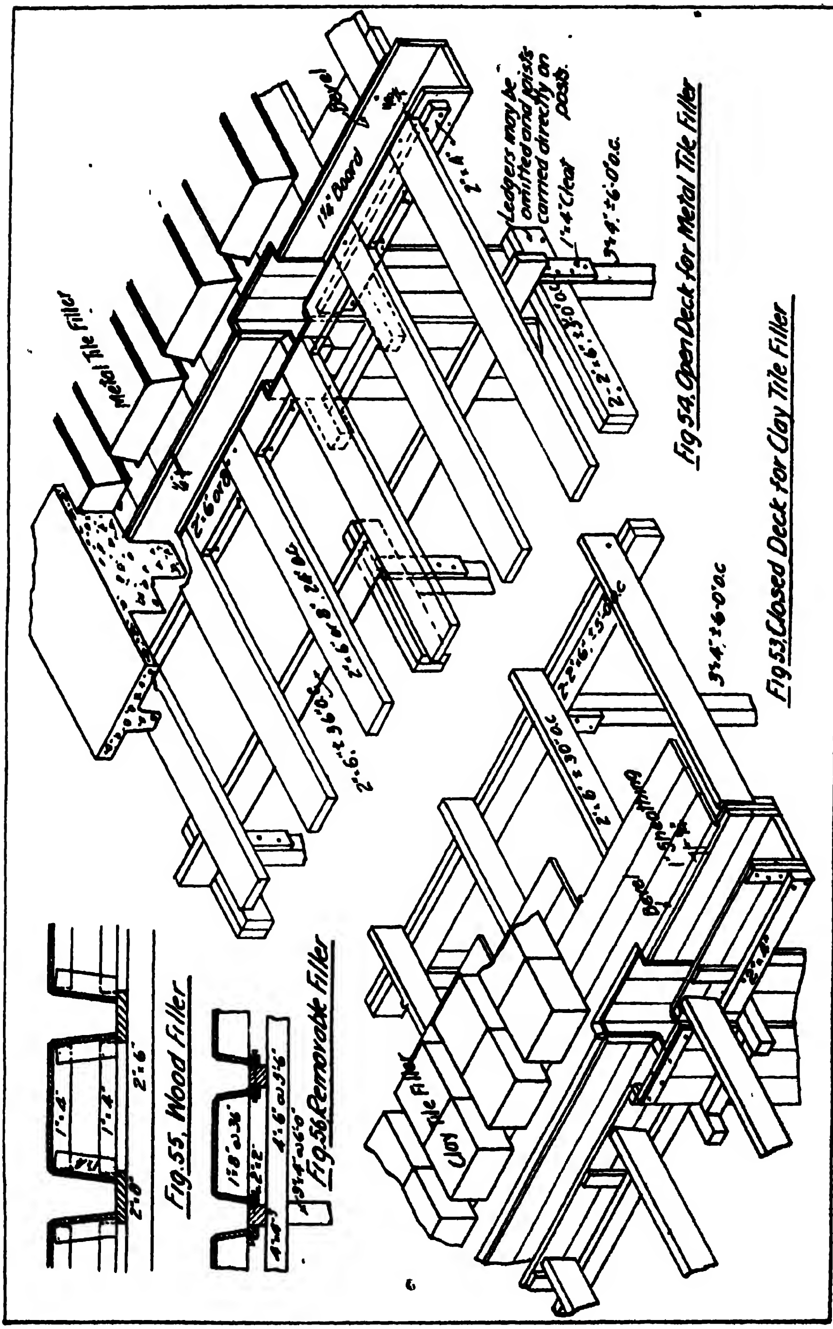


Fig. 54, Open Deck for Metal Tile Filler

Fig. 53, Closed Deck for Clay Tile Filler

Fig. 55, Wood Filler

Fig. 56, Removable Filler

open decks as for closed decks. The planks should preferably be one length, but may be spliced if necessary with cleats on the underside.

Ledgers and posts can be erected separately or together in bents of three or four posts, holding them temporarily at the top with a joist. The filler will nearly always stop a few inches from the beam sides to provide a solid concrete Tee of the total depth of the slab. To provide for this Tee, short pieces of plank, an inch or so wider than the width of the Tee, have to be inserted between the rib bottoms, and nailed to them with heads left projecting. The short pieces should all be ready, cut to exact length, and can be used as spacers for the planks, placing a plank and short piece alternately. If the short pieces are placed after the planks there may be some trouble and time wasted in fitting them in. One or two heavy nails will hold the plank to the beam ledger and a nail at every other joist will hold the joists in place.

The plank can be carried on the beam sides, as in *Fig.* 53, that is, resting on the top with ½ in. clearance. A better construction is shown in *Fig.* 54, where the plank butts against the sides and is ½ in. higher, with the beam sides bevelled. It is easier to bevel the beam sides than the plank The beam sides are held at the bottom with spikes, as they are never very deep. The sides are often not over 8 in, so one board can be used. Joists generally cover three spans, or are about 16 ft. long.

A method sometimes used (though it is not so good) is to run the joists under the centre of and parallel to the planks. The plank and joist are nailed together to form a unit, and the plank rests on top of the beam sides, being about 1 in. longer each end than the joist.

The above remarks on stripping apply also to the open deck. The short filler planks should have the nails drawn before stripping, to free the planks. When reshoring the ledgers should be placed at right angles to the ribs, and the posts must be blocked up from the floor. Ledgers may be omitted, and the joists carried directly on the posts. About the same amount of timber will be required, but there will be twice as many posts to place, plumb, and wedge There is an advantage in that the joists only have to be levelled, instead of both joists and ledgers.

Wood Fillers. (*Fig.* 55)—Instead of steel removable tiles, or when the spacing of the tiles is too great for the standard widths, wood forms are often used. They may be economical on a small job, where only used once or twice, but if they are to be used several times steel tiles are nearly always cheaper as there are no repairs and they have almost full salvage value.

The wood form is made of 1 in. stock nailed to 1 in. frames, forming panels 8 ft., to 10 ft. long. The sides forming the ribs and the ends should slope at least 1 in. from top to bottom in order to facilitate stripping. The frames should be about 36 in. apart and made of 1 in. by 4 in., or 1 in. by 6 in., or they can be made solid with battens. A few diagonal braces are useful to prevent buckling. The frame is supported in the same way as for metal tile fillers. There may be some difficulty in

stripping the forms, as they are liable to stick unless made with a hinge at the top of the frame, which adds greatly to the labour cost, and is not worth while unless the forms are carried as more or less permanent plant. It will help stripping to have the two end top boards narrow and loose.

A method of building the forms so that the filler can be stripped without disturbing the supports is shown in *Fig.* 56. The filler joists in this case are single boards of the depth required; if a stock size will not suit the next widest board can be used and the ends sized down. The joist bottoms are 4 in. by 4 in carried on 4 in. by 6 in. ledgers. On the sides of the 4 in. by 4 in.'s are nailed 2 in by 2 in. strips—using double-headed nails -which support the fillers. Removing these strips enables the fillers to be removed without disturbing the joist supports. The length of the fillers must be less than the distance apart of the ledgers, say about 3 ft. to 4 ft.

Removable Metal Tiles. These are economical for high buildings or very large areas, when they can be used at least four times over, or when for architectural reasons a beam ceiling is desired.

Fig. 57 (*a*) and (*b*) shows two methods of building the forms when the joists are close together, say about 24 in. So that the tiles will be economical for use on different jobs, they are made deep enough to suit several different depths of joist The joist support is a 1 in. by 4 in., or whatever width of joist is specified, nailed to a 2 in. plank on edge. The tiles overlap the joist bottom and at (*a*) are carried on 1 in. by 2 in. about 18 in apart supported by 1 in. by 2 in. ledgers nailed to the plank. Spacers of 1 in by 2 in. will prevent the tiles spreading at the bottom. To strip, the 1 in. by 2 in supports are knocked off the ledgers and the tiles are sprung out The tiles are made in about 3 ft lengths. At (*b*) the ledgers are 2 in. by 2 in, and 1 in. by 1 in. ribbons are used to hold the bottom of the tiles In either case the plank can be supported directly by posts or by ledgers to reduce the number of posts.

For longer spans between joists the method shown in *Fig.* 57 (*c*) can be used. The joist support is a 2 in by 6 in. nailed to the top of 4 in. by 4 in. posts. The tiles are carried on 2 in. by 4 in.'s supported on continuous 1 in. by 6 in.'s nailed to the posts The tiles are also nailed —through holes supplied for that purpose to the joist bottom. Double-headed nails are preferable whenever they are to be pulled before stripping.

Usually one-and-a-half or two floors of steel tiles are used, so that there is always one floor fully formed beneath the floor being poured. The floor that is stripped should be reshored. The metal tiles can be removed in two to three days for the short spans and four to six days for the long spans.

Two-Way Joist Construction.—In simple joist construction the joists run in one direction, being carried by two beams or walls. If it is desired to distribute the slab load more equally a "two-way" construction is used; that is, the joists run in two directions at right angles, and are supported on four beams or walls. The fillers then, instead of being continuous rows, will be square units, or "domes" as they are generally called.

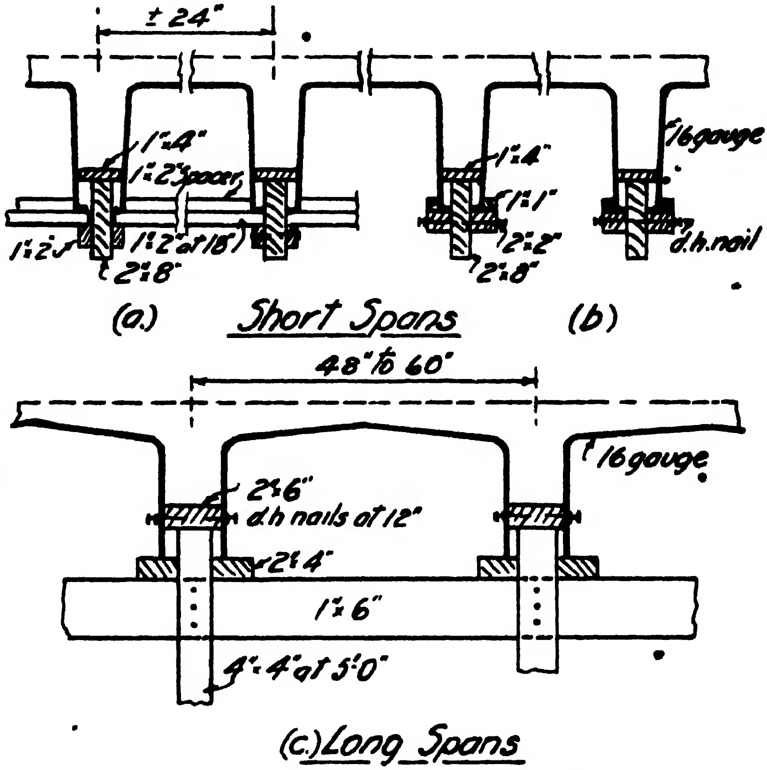


Fig. 57, Removable Metal Tile

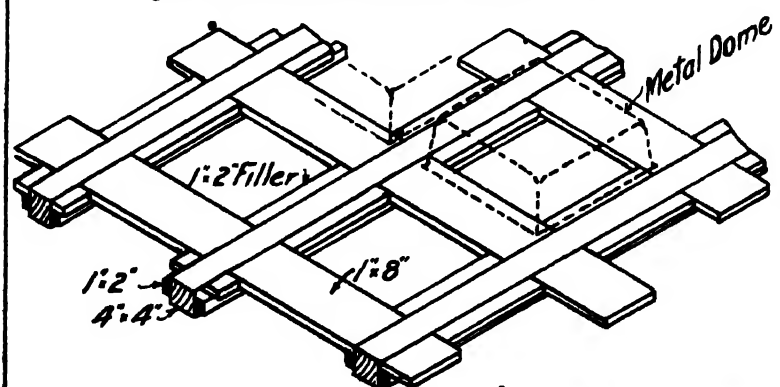


Fig. 58, Two-Way Joists with Metal Domes

This construction is more common with steel fillers than with hollow tile because closed ends are required, while the ordinary hollow tiles have open ends. If hollow tiles are used the construction is the same as the closed deck already described. The load to be carried will be slightly greater, as there will be more concrete, so the supports must be a little closer.

Metal tiles, or "domes," about 20 in. square at the base are supported on open deck forms, but the construction shown in *Fig.* 54 is not easily adapted for domes because the joist bottoms running in one direction must support the bottoms in the direction at right angles, and this is not easily done with 2 in. plank.

[*By permission of The Trussed Concrete Steel Co.*

FIG. 59.—OPEN DECK FORMS FROM ABOVE, SHOWING "FLORE-TYLE" AND "HY-RIB" LATH.

Instead, the construction shown in *Fig.* 58 can be used. The joist bottoms in one direction are 4 in. by 4 in.'s. To each side of the 4 in. by 4 in., and 1 in. down from the top, is nailed a 1 in. by 2 in. ledger. This ledger carries 1 in. by 8 in.'s at the same distance apart as the 4 in. by 4 in.'s, forming the bottoms for the joists in the opposite direction. On top of the ledgers, in between the 1 in. by 8 in.'s, are nailed 1 in. by 2 in. fillers, carrying the domes. The 4 in. by 4 in.'s are supported as in *Fig.* 54, but the supports must be closer together because of the greater dead load.

An alternative to this method is to substitute for the 1 in. by 8 in. joist bottom a 16-gauge iron plate the width of the joist and supported by the dome flanges. In this case the ledgers are 2 in. by 2 in., nailed flush with the top of the 4 in. by 4 in., and are continuous, no filler strip being required.

[By permission of The Trussed Concrete Steel Co.

FIG. 60.—FORMS FOR REMOVABLE METAL TILE

[By permission of The Trussed Concrete Steel Co.

FIG. 61.—FORMS FOR WIDE-SPAN REMOVABLE METAL TILE.

With either construction the domes can be removed without disturbing the supports. *Figs.* 59, 60, 61, show construction of forms for fixed and removable metal tile fillers manufactured by Trussed Concrete Steel Co.

Estimating Cost.—Closed deck forms will require about ¼ cu. ft. of timber per sq. ft. of floor area, and open deck forms about ⅕ cu. ft. per sq. ft. for average conditions. Open deck construction therefore requires about 20 per cent. less timber than the closed deck. These amounts include posts, bracing, waste, etc. The labour cost is about the same for either method, since with the former there is less timber, but the work has to be done more accurately, and with the latter there is more timber to handle but erecting it is simpler. There will, however, be more salvage of material with the heavier timber of the open deck.

A full set of forms should be estimated for one floor or a floor and a half, and these can be used again on upper floors.

To frame and erect forms for 100 sq ft of floor will require about 5 hours carpenters' time and 5 hours labourers' time, and to strip them about ½ hour carpenters' time and 2 hours labourers' time.

Cost of forms for 100 sq ft of floor (open or closed deck):—

5½ hrs carpenter @ ......
7 ,, labourer @ ......

For closed deck, timber required = 100/4 = 25 cu. ft.
,, open ,, ,, ,, = 100/5 = 20 ,, ,,

Cleaning and hoisting will each require about 1 hour labourers' time.

Beams can be estimated as in beam and girder construction, also reshoring.

The cost of wood fillers should be estimated separately from the forms and be compared with the cost of tile or metal fillers. They will require about ⅛ cu. ft. of timber per sq. ft. of floor area, and it will require about 5 hours carpenters' time and 2 hours labourers' time to make up sufficient fillers for 100 sq. ft. of floor.

The cost of forms for wood fillers and removable metal tile is about the same as for the open deck in both material and labour.

Forms for two-way joists will cost about 10 per cent. more in material and labour per sq ft. of floor area than the open deck.

Structural Steel Fireproofing.

The forms for supporting the concrete fireproofing around a structural steel frame (*Fig.* 62) are built much simpler and lighter than for reinforced concrete frames. Cinders as aggregate instead of stone are used to cut down the dead weight of the structure, forming a concrete weighing only about 100 lbs. per cu ft., or about two-thirds the weight of stone concrete. Little or no live load need be allowed for in designing the forms, as wheeling of concrete is done on runways supported directly by the steel beams.

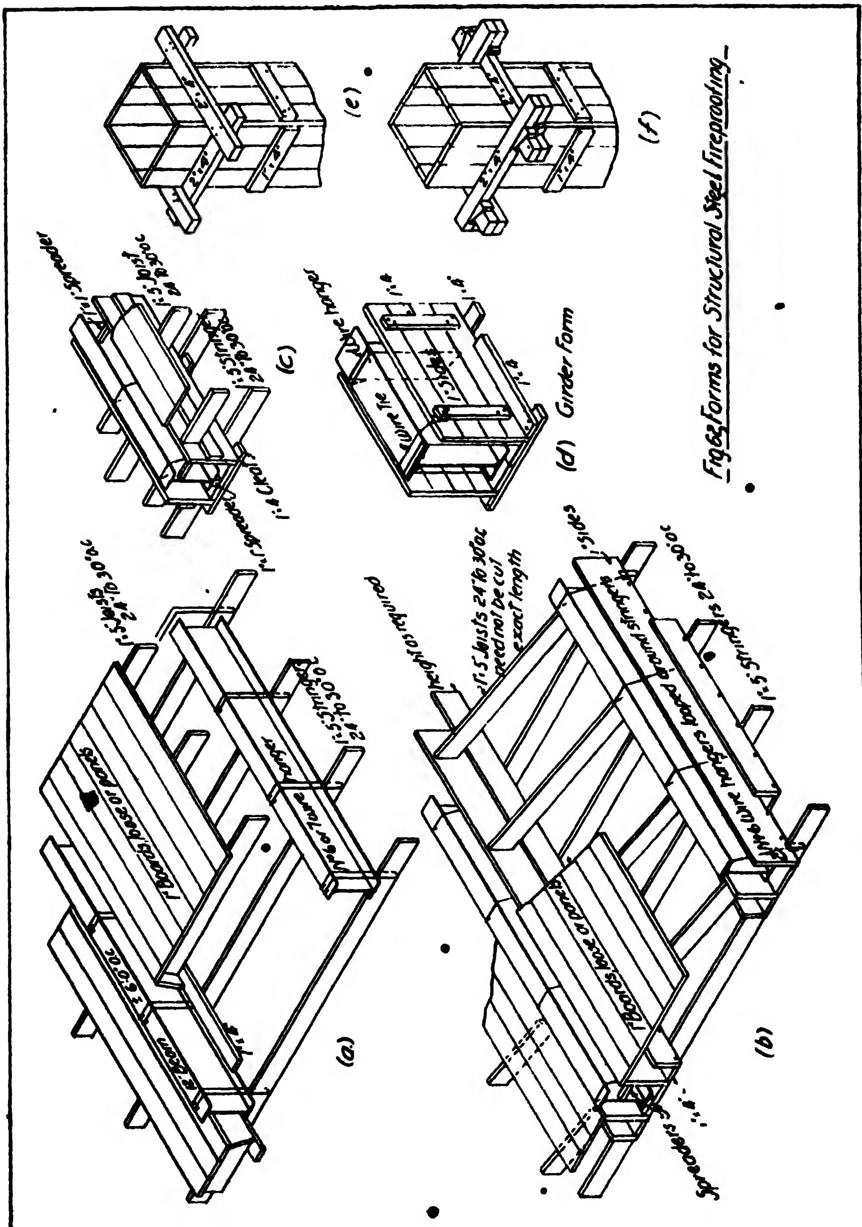


Fig. 62. Forms for Structural Steel Fireproofing.

As there is practically always a suspended ceiling beneath the floor and other exposed concrete is plastered, the lines and appearance of the concrete are not so important, so that strength in bending rather than allowable deflection will govern the form design.

The floor forms are always suspended from the steel beams, eliminating all wood posts, so that the forms do not have to carry the floor above as in reinforced concrete construction. Dressed boards, nominally 1 in. thick, being the cheapest timber to buy, are used practically throughout. Often only one width of board is necessary, being mainly governed by the depth required for the joists. The usual widths are 4 in., 5 in. or 6 in., dressed about ¼ in., with square edges. Having only one size to deal with greatly simplifies the timber list and reduces the labour. Hangers consist of heavy bright wire number 6 or 7.

The work of erection should be subdivided into placing hangers, beam forms, stringers, etc. The general method is to carry the beam and slab forms by continuous stringers hung from the top flange of the beams.

The floor slab is usually 4 in. thick, of cinder concrete, and will weigh about 35 lbs. per sq ft. The span between beams is generally between 6 ft. and 8 ft. The safe carrying capacity of 1 in. joists, depending on their strength in bending for various spans and spacing, is as follows :—

						lbs. per sq. ft.
For 5 ft. span,	1 in. by 4 in.	at 30 in.	on	centre	will carry	25
		at 24 in.	,,	,,	,, ,,	35
,, 6 ft. ,,	1 in. by 5 in.	at 30 in.	,,	,,	,, ,,	30
		at 24 in.	,,	,,	,, ,,	40
	1 in. by 6 in.	at 30 in.	,,	,,	,, ,,	45
		at 24 in.	,,	,,	,, ,,	55
,, 7 ft. ,,	1 in. by 6 in.	at 30 in.	,,	,,	,, ,,	30
		at 24 in.	,,	,,	,, ,,	40
,, 8 ft. ,,	1 in. by 6 in.	at 24 in.	,,	,,	,, ,,	30
		at 18 in.	,,	,,	,, ,,	35

The span of the joists is the span between supports, not centre to centre of the beams.

The stringers can be the same size and spaced similarly to the joists. The slab boards are either laid loose or built into panels. The beam sides, where possible, consist of one width of board, but for deep beams two or more boards must be nailed together with battens.

Beam bottoms are also 1 in. thick and generally single boards. Girder bottoms are usually wider, and require two boards with battens. Wire hangers should be number 6 or 7 bright wire, not black wire. The sequence of erection is wire hangers, stringers, beam and girder forms, joists and slab panels.

At (*a*) *Fig.* 62 is shown the construction of the forms when the under-

side of the beams is not fireproofed. It is best suited for shallow beams, 10 in. or less in depth, so that a 5 in. or 6 in. joist can be used. This system can be conveniently built in panels 6 ft. to 8 ft. long, nailing the joists, beam sides, and slab boards together. The panels are then carried by 1 in. by 4 in.'s resting loosely on the stringers. The beam sides should be battered for easy stripping.

Wire hangers should be cut to required length and bent before placing. The stringers are pulled up tight against the lower flange of the beam, and the ends of the hanger hammered tight up around the top flange. In this case both legs of the hanger are shown carried over the top flange,

[*By permission of the Thompson-Starrett Co.*

FIG. 63.—SUSPENDED FORMS FOR STRUCTURAL STEEL FIREPROOFING, EQUITABLE BUILDING, NEW YORK.

but the more usual construction is shown at (*b*). The length of the hooks under the top flange should be about 4 in.

It is more usual to encase the entire beam in concrete, and the best construction for the forms is shown at (*b*), *Fig.* 62. In this case the joists are carried on L-shape supports, resting loosely on the stringers; the horizontal leg may be 4 in. to 6 in. and both legs are nailed together. The advantage of this method of joist support is that the span of the joist is reduced, and the joists need not be cut to the exact distance between beam sides as is necessary when they are carried by ledgers nailed to the sides. By placing the joists on a skew their lengths can vary an inch or so and they can be wedged against the sides, thus keeping them in place without nailing and also holding the sides from bulging.

The slab boards may be loose or made into panels, in which case the two outside boards should be loose.

The usual construction for the hangers is to loop one end over the stringer and the other end over the top flange, hammering it well under the flange. The hangers pass between the bottom and side forms. Spreaders 1 in. by 1 in. should be used at intervals to space the forms from the bottom and sides of the steel beam.

Stripping in either method of construction consists merely in cutting the hangers and lowering the forms.

If the beam is so deep that two ordinary width boards are not sufficient to form the joists and vertical leg of the joist support, the joists are then carried as at (*c*) on a ledger nailed to the sides. The girder form shown at (*d*) consists of side and bottom panels made up with 1 in. by 4 in. cleats, 24 in. to 30 in. apart. The wire hangers pass through and under the bottom cleats and the legs are vertical, and are bent around the top flange. The holes in the cleats should not be in a line parallel to the sides, but one hole should be near one edge and the other hole near the opposite edge; that is, the wire passes diagonally under the cleat. The top of the girder is held from bulging by wire ties around the side cleats, or it can be braced back diagonally to the beam sides. If the joists are carried on girder sides the construction will be as at (*c*.)

Column forms are built much lighter than in reinforced concrete construction, as the pressure will be much less. Generally the sides are panels of 1 in. boards cleated together every 24 in. to 30 in. The yokes are loose and can be 2 in by 4 in nailed together as at (*e*), or wedged together as at (*f*) *Fig* 62. The latter method requires more labour cutting and fitting the wedges and stop blocks, but is much easier to strip. Patent adjustable column clamps are often used in place of wood clamps.

Connection of beams to girders and girders to columns can be made as previously described, using 1 in. stock instead of 2 in.

Stripping of beams and girders can be done in two to three days, and slabs in three to seven days

If stone concrete is used instead of cinder concrete the same methods of construction can be used, although since the dead load will be greater all the forms must be correspondingly heavier, and instead of wires, bolts will be required hooked on the lower or top flange with washer and nut under the beam bottom cleat.

Estimating Cost.—From $\frac{1}{5}$ to $\frac{1}{4}$ of a cu. ft. of timber should be allowed per sq. ft. of total contact area, including slab, beams, and columns; this will include waste. In high buildings sufficient timber should be provided to form from two to three whole floors.

The labour cost will vary greatly with the experience of the contractor in this class of work and with the size of the job and speed required. It will vary from as high as the cost of forms for reinforced concrete beam and girder construction, as given in Chapter IX, to about 25 or 30 per cent.

less than this. In America this work is largely handled by firms who specialise in fireproofing structural steel as sub-contractors, doing no other part of the work. Since these firms have experienced men who do nothing but build forms for structural steel fireproofing they can obtain much lower costs than the general contractor, the average cost being about 30 per cent. less than that given in Chapter IX.

CHAPTER XI.

MISCELLANEOUS FORMS IN BUILDING CONSTRUCTION.

Stair Forms.

STAIRS are designed to be self-supporting longitudinally from floor to floor or floor to landing, or across from wall to wall. The former method is more common when the whole frame of the building is concrete, and the latter when the floors only are concrete supported by brick bearing-walls.

The stairs may be a straight run from floor to floor (*Fig.* 64), but more often there is an intermediate landing (*Fig.* 64, dotted lines). Usually the stairs are not poured until after all the floors are completed, making this a separate operation from ground floor to roof. This saves time in pouring the main floors.

With straight runs from floor to floor ledges and keys are left in the floor beams to support the stair slab.

In order that the risers in each flight may come vertically over each other it is best to leave the stair opening a few inches wider than actually required to allow for adjustment horizontally; this clearance is shown as 6 in. in *Fig* 64, 3 in. top and bottom. If the opening is made the exact size it is difficult to carry the edges up plumb throughout a high building.

The sloping slab is supported on 4 in. by 4 in. joists and ledgers. The posts should preferably be as nearly at right-angles to the ledgers as possible, with wedges between, but may be placed vertical if cleated to the ledgers and well cross-braced.

The side forms, or stringers, are cut out of 2-in. plank from 10 in. to 12 in. deep, as required. To these are nailed the riser forms, also of 2-in. plank.

On straight runs the stringers are cut as shown in *Fig.* 66, the dotted lines indicating the full plank and the solid lines the cuts. Starting from one upper corner of the plank, the length of the tread and riser is marked out successively, the top of each riser being on the edge of the plank. The clearance allowed—this should be measured from the actual opening—is then marked off and the plank sawn through parallel to the risers.

When there is a landing the clearance will only occur at the floor line, and at the landing the stringer ends at the junction of the stair and landing slabs, and a plank the depth of the landing slab is cleated on.

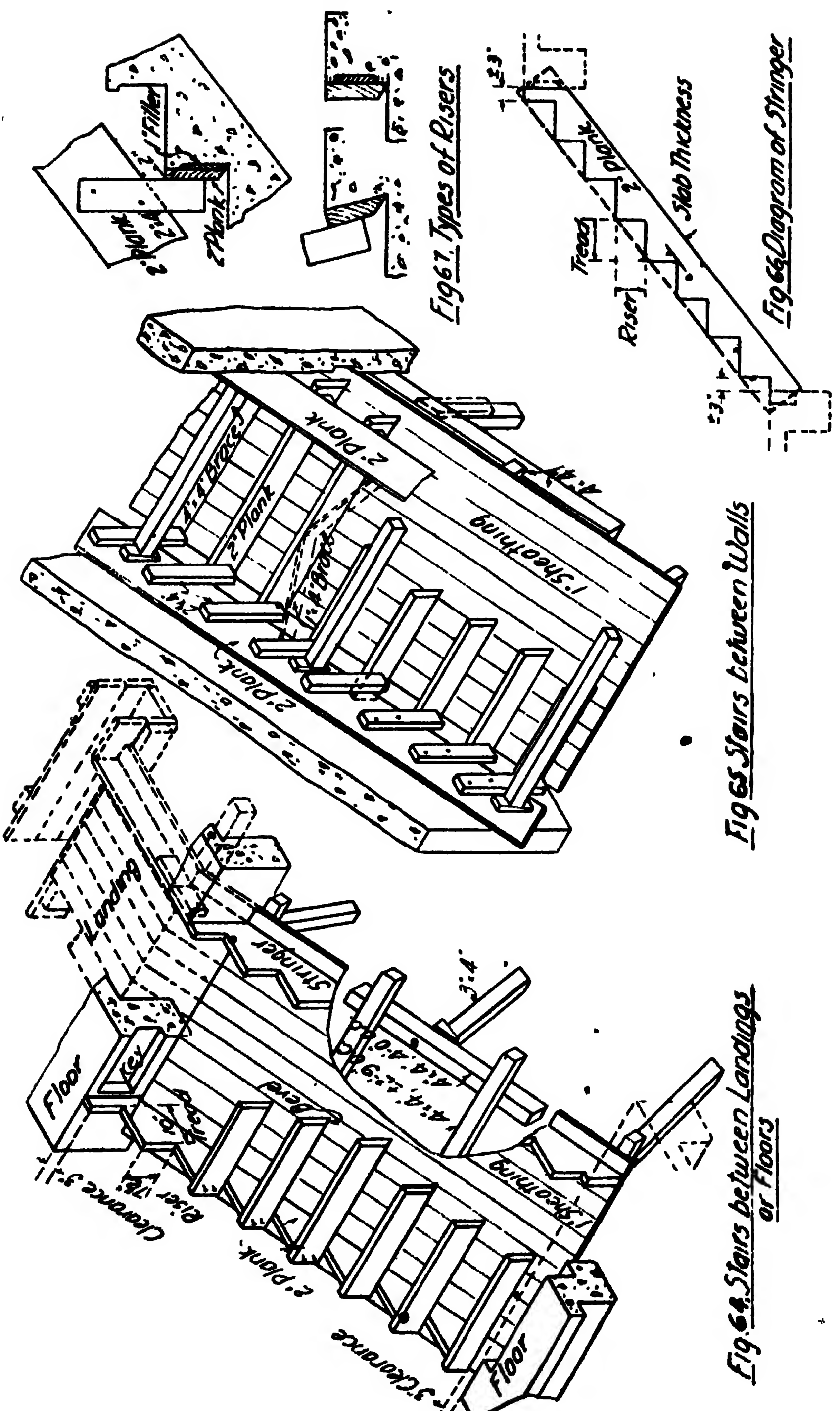


Fig. 64. Stairs between Landings or Floors

Fig 65 Stairs between Walls

Fig 66 Diagram of Stringer

Fig 67. Types of Risers

The stringers are set in position on the sheathing, and may be nailed in place or held by a 1-in. ribbon.

The whole form is wedged up to the correct height. When there is a landing there should be a joist at the junction of the slabs.

The landing sheathing is carried on the beam side with intermediate joists as required by the span and thickness of the landing. The landing beam is usually poured with the stairs, being supported either by brackets built on the columns for that purpose, by independent columns, or by hangers from the beam above at the floor.

On the face of each stringer is nailed a 2-in. plank to form the riser, and of the same depth as the riser. The bottom of these planks should be bevelled so that the finisher can get his trowel into the corner, otherwise there will be a ridge left on the concrete. If each tread has a slight slope for drainage, this slope must be allowed for in laying out the stringer.

The risers may slope inwards, may be recessed, or have nosings, as shown in *Fig.* 67. To form these nosings a 1-in. filler-piece is nailed to the riser plank.

If the stairs are wide and there is danger of the riser planks bulging, a long plank or 2 in. by 4 in. can be placed on edge across the top of the risers nailed to posts wedged between the floors. For very wide stairs one or more intermediate stringers can be used inverted over the riser planks and held in the same way, or the planks can be wired back to the under form.

When the stairs are supported between walls already built (*Fig.* 65) the slab form will be the same as before, or if built on a fill no form is necessary. Sloping ledges are left in the wall to give bearing to the slab.

The riser forms are supported differently, as they must be hung from above. Two 2-in. planks are placed along the walls so that they will clear the risers by 2 in. or 3 in. They are braced together by 4 in. by 4 in. and wedged, or they can be supported by 4 in. by 4 in. posts at each end wedged between floors, or the planks can be bolted to the walls.

To the planks are nailed 2 in. by 4 in. vertical hangers at distances apart equal to the width of tread. The hangers should stop 2 in. or 3 in. above the treads. To the hangers are nailed the riser planks, with fillers attached when required. The risers must be cut the exact length between walls.

Stripping the risers can be done in one to two days, but it is advisable to leave them longer to protect the edges.

Cost.—The cost of stair forms can be estimated by the flight, by the square foot of under surface, or by the lineal foot of riser; the two latter methods give about the same result since there is usually about 1 sq. ft. of surface to 1 lineal foot of riser. An average flight from floor to landing will need 40 to 50 sq. ft. of forms, measured along the slope, for a 12 ft. story height and 4 ft. to 5 ft. width of stairs.

If the under-form is supported, each square foot will require about ½ cu. ft. of timber, including stringers and risers. If the stairs or steps

are built on a fill, no under-form being required, about $\frac{1}{3}$ cu. ft. per sq. ft. should be allowed.

The labour cost is best estimated by the flight from floor to landing. An average flight, as above, will be a day's work for two carpenters assisted by about $\frac{3}{4}$-hour labourers' time to each hour carpenters' time, including erection and stripping

Cost of 1 flight of 40 sq. ft.—

16 hrs. carpenter @	=	
12 hrs. labourer @	=	
20 cu. ft. timber @	=	

The timber can be used several times over, so that the above amount can be divided by the number of times used.

For longer or wider flights the cost can be estimated proportionally by reducing to a square foot basis.

Cornice Forms.

Cornices are of so many different designs and shapes that it is only possible to describe the main points in the construction of the forms, the actual details being different with each design. If the cornice is heavy and complicated it is always built separate from the roof slab, the wall beams being keyed to receive it. Only the simplest of designs is built with the roof slab, because if it is at all complicated the time required to make and place the forms would hold up the pouring of the roof; and also, since the forms are expensive to make, it is usual to make up a minimum length and use it over several times.

Generally the wall beams are stripped and reshored, but the outside and bottom forms may be left on if they are of any assistance in supporting the cornice form.

The method of shoring the cornice is very important, as most of the weight will overhang the building. They should be placed closer together than required for strength, generally from 3 ft. to 4 ft. As the posts cannot be placed vertically under the load there will always be some bending action, so that at least two posts should be used and well-cross-braced together. The shores will be what are known as "outrigger shores," the outside arm being longer than the inside arm. They consist of two or more posts, cap and braces.

Various types of outrigger shores are shown in *Fig.* 68. The type to use will depend mainly on the design of the cornice. If the cornice is vertical (that is, if most of the weight comes on the roof), type (*a*), (*c*) or (*d*) can be used. If the cornice overhangs, types (*b*) and (*e*) would be better. The main points to remember are that the posts will tend to be pulled outwards at the top and pushed inwards at the bottom. When a vertical and inclined post butts at the bottom, as at (*a*) and (*e*), kick blocks should be bolted to the floor to take the thrust. At (*e*) is shown a good form of shore, where the inside post is wedged against the roof slab, and the cap consists of two planks nailed to the sides of the posts.

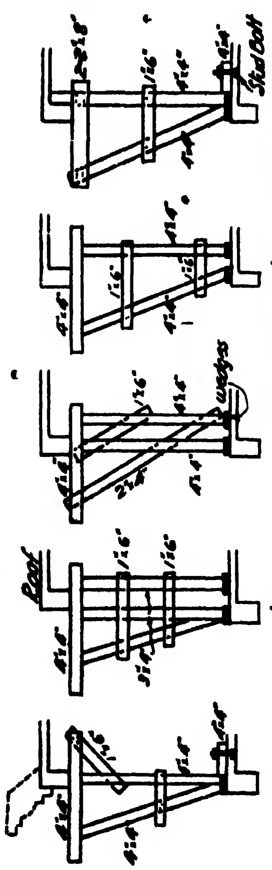
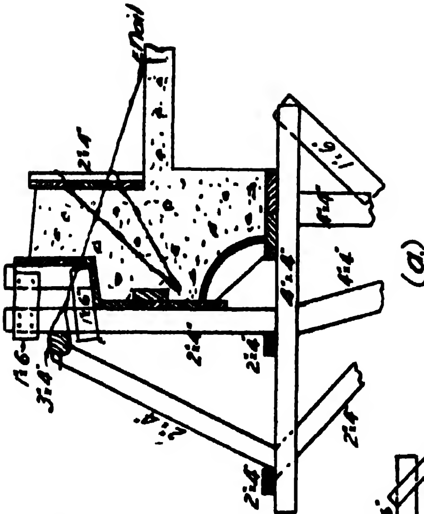


Fig 68. Types of Outrigger Shores for Cornices



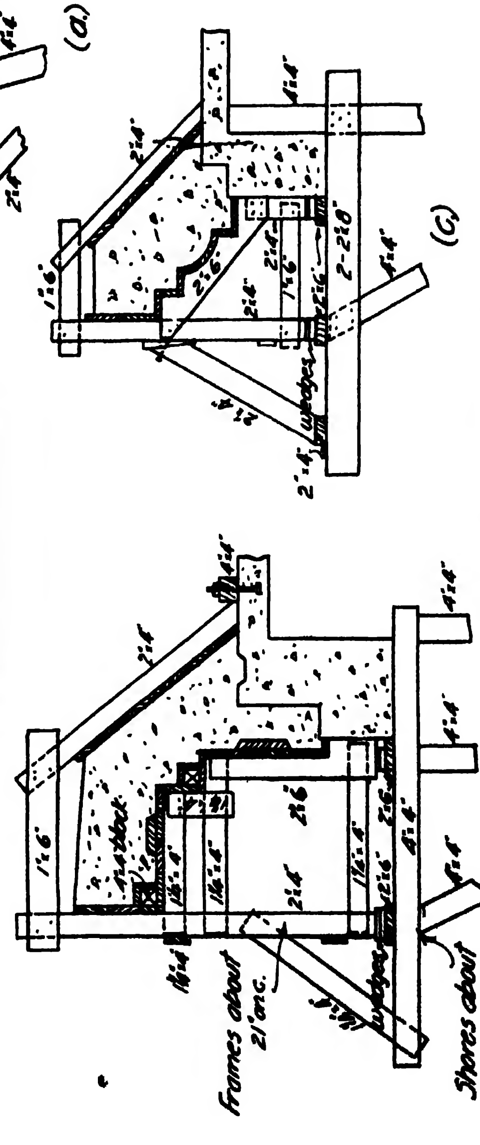


Fig 69. Cornice Forms

Batter posts should be under the point of application of the load, so that there is little bending in the cap. All posts should be on wedges.

On top of the shores are set the built-up frames to which is nailed 1 in. or 1¼ in. sheathing. The type of frame will vary with the design of the cornice, but is generally made up of 1¼ in. or 2 in. by 4 in. nailed together and braced. At (*a*), (*b*) and (*c*), *Fig.* 69, are shown typical designs. When there is only one vertical leg to the frame, as at (*a*), it should be braced from the cap and tied back to the roof. In this design the curved portion is a separate unit lightly nailed to the side and bottom. If the form is to be stripped quickly the piece forming the inset should be nailed on lightly so that it will remain in the concrete and protect the edges in stripping. A 3-in. by 4-in. wale should be used to hold in line the uprights, and when there is any pressure on the inclined brace to the wale the end of the cap should be braced back to the posts.

When the frames are wide, as at (*b*) and (*c*), wales need not be used, but diagonal braces are advisable to prevent any tendency of the form to be pushed outwards. In designs such as (*b*) and (*c*) it is best to set the frames on wedges so that they can be lowered to clear the concrete before swinging them out; this will prevent corners being broken off.

When there are undercuts or recesses in vertical faces, as in (*b*), the boards to form these should be nailed on lightly to the form, so that they will remain in the concrete when stripping. If the nailing is done from the outside, leaving the heads projecting, the nails can be withdrawn leaving the inset board loose to enable the form to be lowered.

At (*b*) the frame is built to take care of the main details of the cornice, and the remaining details are blocked out with independent forms of sheathing nailed to 4 in. by 4 in. blocks. This method should be used whenever possible, as it makes a strong form and there is no expensive labour cutting members to the exact outline of the cornice.

At (*c*) the template method is used, the 2 in by 6 in. joists being cut to the outline.

Frames should be 18 in. to 24 in. apart, depending on the weight to be carried. There should be shores at alternate frames. Intermediate frames are supported by 2-in. continuous planks, unless the pressure is mostly horizontal, as at (*a*), when no support is necessary. Some longitudinal bracing is necessary to add stiffness to the form.

The complete form is usually built up on the ground or on the roof in lengths of 6 ft. to 10 ft., depending on the weight. A light hand-derrick is necessary for placing the forms and handling them while stripping. The back-form of the cornice can be held in several ways, as shown in *Fig.* 69. Wires anchored around the reinforcing steel in the concrete are useful for this purpose.

The length of time the forms should be left before stripping depends on the overhanging weight. Type (*a*) could be stripped in two to three days, while (*b*) and (*c*) should be left a little longer. When the forms

are to be used several times a short length can be stripped and immediately reshored from the main shores.

When there is a moulded belt-course running around the building the forms for it can be built in the same way as for cornices. If it is simple it will be cast with the floor, but if elaborate will usually be built afterwards, keys being left in the concrete to support it.

Cost.—Cornice and belt-course forms can only be estimated by experience, since they vary so much in design, but they will cost from two to four times as much as ordinary beam forms per sq. ft. of contact area.

Sill Forms.

Concrete sills are poured after the walls are built and the steel window frames set, as they hold the bottom of the frames. The form (*Fig.* 70) is built in two halves, each consisting of a plank with 2 in. by 4 in. uprights attached to form a clamp. A 2 in by 4 in. cross-piece nailed on to the top of the uprights holds them the right distance apart and a bolt draws them tight against the wall, with spreaders to give a uniform width of sill. The outside plank will have a strip nailed on the bottom to give the overhang, and to this strip is nailed the drip-moulding, which may be triangular or half round This drip-moulding should only be nailed on very lightly, as it should stay in the concrete until it can be taken out without breaking off the outside edge of the sill. The sills can be stripped within two days.

Cost.—The unit cost of sills is usually estimated by the lineal foot of sill, which is about equivalent to 1 sq. ft. of contact area. A lineal foot will require about $\frac{1}{8}$ cu. ft. of timber, but the timber can always be used several times. The labour required to place and strip will be about 10 minutes carpenters' time plus 10 minutes labourers' time per lineal foot of sill.

Upturned Beams.

Exterior wall beams are often turned up to give more light to the building (*Fig.* 71). The beam and slab should be poured together, so the inside form of the beam must be supported above the slab. The side-form is built in the ordinary way, but a 2-in. plank is nailed on the bottom ; otherwise the concrete will come up on the inside and make it difficult to strip. This plank must be held down against upward pressure, and this can be done with wires anchored to the reinforcing steel with wedges to take up the slack. To support the form above the slab, temporary blocks of wood can be used and knocked out during concreting, or concrete blocks can be left in the slab. The outer end of the joists can be supported on two 2 in. by 6 in., separated as shown, using short intermediate joists if necessary to cut down the span of the sheathing carrying the extra weight of the beam.

Cost.—These beams will cost more than interior beams, and an allowance of about 25 per cent. more per sq. ft. can be made.

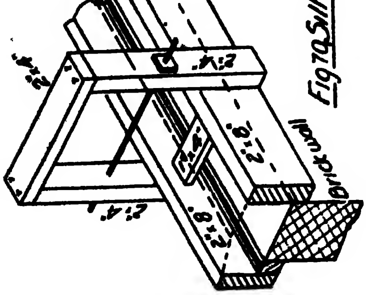


Fig 70 Sill Form

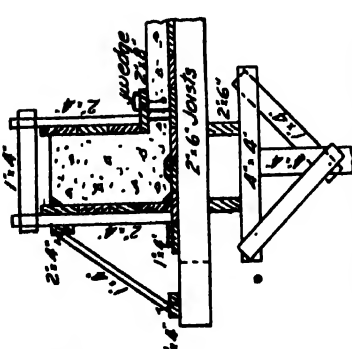


Fig 71, Upturned Beam

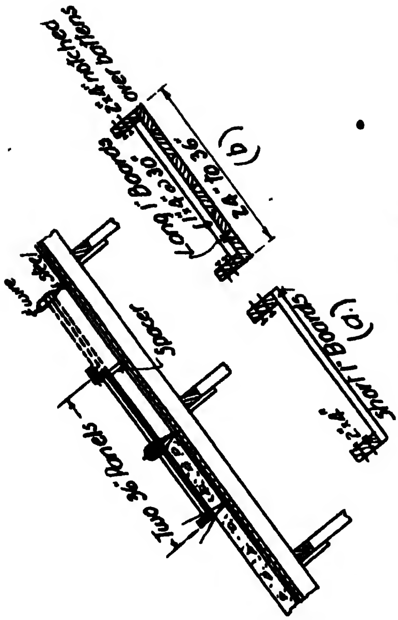


Fig 73, Backforms for Sloping Slabs

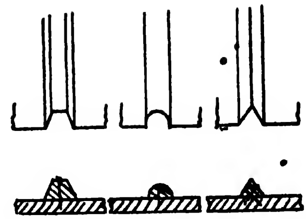


Fig 72, Column Marking

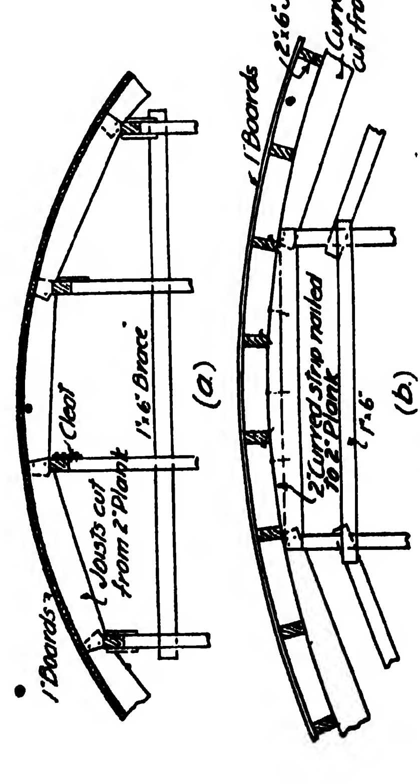


Fig. 74. Curved Slab Forms

Column Mouldings and Ornaments.

Columns are sometimes marked out to make the concrete resemble cut stone work by nailing to the main form narrow strips of mouldings of the desired shape (*Fig.* 72). These strips should always be bevelled for easy stripping. Nailing and mitering the corners of the strips must be done very carefully for satisfactory results. The strips should preferably be nailed on lightly so that they will remain in the concrete, otherwise some edges are sure to break off if the strips and forms are removed together while the concrete is still green, and early stripping is desirable so that the concrete can be finished. With very high columns the strips, if nailed on lightly, may become dislodged during concreting, so in this case should be more securely nailed and the forms left on longer. The forms should be well oiled.

Inset ornaments and slots are made by cutting boards to the desired shape and nailing them to the main form.

Projecting ornaments, especially if they are elaborate, are generally more easily handled by precasting and setting the finished moulding in recesses in the concrete or by setting in the forms and pouring around them.

Cost.—The cost of ornamental work can vary enormously, and can only be estimated by experience. Any elaborate form should be made in a joiner's shop, and definite quotations can be obtained.

Back-forms.

When a concrete slab is poured on a slope, as, for instance, for a saw-tooth roof, a top or back-form (*Fig.* 73) is required to hold the upper surface of the concrete. It is hardly possible to pour concrete on a slope greater than thirty degrees with only an under-form, and even on this slope great care is required to prevent the concrete from running down the slope. It is not practical to build the form the full height before concreting, as it would be almost impossible to pour a thin slab from the top without getting numerous voids.

The form is best built in panels 2 ft. to 3 ft. wide and as long as the slab, or if this is too great in about 12 ft. lengths.

Using two panels, the first is placed and wired down to the reinforcing steel, using 1 in. sq. spacer blocks to hold it above the lower form ; then the form is filled to the top and the next panel is nailed on, wired down, and filled. The lower panel is then removed after drawing the nails and cutting the wires, moved ahead of the second one and so on. Using three panels the concrete is poured to the top of the third before removing the lowest one, allowing a little more time for the concrete to set.

In warm weather a panel can be removed in an hour, but the concrete should not be trowelled immediately as this would cause a tendency to flow.

The panels can be made most economically with the greatest salvage of material by running the boards lengthways in long lengths with 1 in.

by 4 in. battens about 30 in. apart, and on each edge is nailed a 2 in. by 4 in. on edge, notched over the battens so that they can be nailed to the boards. These 2 in. by 4 in. act as wales and for nailing the panels together (*Fig.* 73 (*b*)). By this method the battens are the only short pieces required.

Another method (*Fig.* 73 (*a*)) is to run the boards the height of the panel with 2 in. by 4 in. on each edge and no battens, but this means cutting up the boards into short lengths with consequent waste.

The wires can be tied around the joists of the lower form, but this is more expensive as holes have to be bored ; and it is not necessary, as the pressure on the back-form is small. There should be a wire to about every 15 sq. ft. The back-forms should be oiled as well as the lower form, as they have to be stripped quickly

Cost.--The cost of back-forms is about the same as the cost of making, placing and stripping slab-panel forms in beam and girder construction.

Curved Slab-forms.

Curved slab-forms (*Fig.* 74) can be built in two ways. Either the joists themselves can be cut to the required radius and supported by straight ledgers, or the joists can be straight and the ledgers cut to the radius. The first method would be better for a sharp curve and small area, and the latter for a flat curve and a large area.

At (*a*) the joists of 2-in. plank are cut as described for curved walls, nailed together, and notched at the joints to give square bearings for the ledgers. The posts should be well-cross-braced to take the side thrust.

At (*b*) the joists are toe-nailed to the curved ledgers to hold them in place. The ledgers should be notched to give the posts square bearing, or the tops of the posts can be shaped, or wedges can be used. Instead of cutting the ledgers they can be left square and curved strips can be nailed on the upper edge. By this means there is full salvage of the ledgers and only the strips are wasted.

In power-house construction there is often the problem of building forms for a slab that is flat at one end and curved at the other, the curvature gradually increasing from a straight line to a maximum curvature. This can be done by using straight joists resting on a horizontal ledger at one end and a curved ledger at the other. The joists are placed at right-angles to the curved ledger. The ends of the joists on the horizontal ledger must be bevelled to give a square bearing. The joists should preferably be heavy and so close that an intermediate ledger will not be required. If a board is laid over the joists it will bear only on one edge of each joist, so that the top of each joist should be bevelled to give full, or nearly full, bearing to the boards. Square-edge boards should be used for sheathing ; as many boards as possible are nailed together, then a space is left afterwards to be filled with a special board, as described for sloping curved walls. The change in curvature in the width of a

board will be so slight the small ridges formed in the concrete will hardly be noticeable.

Cost.—Curved slab-forms will require about $\frac{1}{3}$ cu. ft. of timber per sq. ft. of contact area, which can be divided by the number of times used. To erect and strip 100 sq. ft. of forms will require about 12 hours carpenters' time and 10 hours labourers' time.

CHAPTER XII.

FORMS FOR FLAT SLAB CONSTRUCTION.

A SYSTEM of floor design which is by far the most popular in America for industrial buildings, but which at present is little known in this country, is the "flat slab" or "mushroom slab" floor. When the many advantages of this system become recognised it will undoubtedly largely supersede the ordinary beam and girder design, as it has done in America for several years.

The formwork is much simpler to build, as there are large flat areas with no beams; the only beams occur along the outside walls and around elevator and stair wells. Over the column there is usually, though not necessarily, a square "depressed panel" or "drop head," a few inches deeper than the slab The interior columns are almost always round, because that is the best shape structurally.

The columns are always finished with cone-shape caps beneath the depressed panel, which may be either plain or moulded. Column caps, when round, are always built with sheet-metal steel forms. Occasionally square and octagonal columns are used, built with wood forms.

The depressed panel form is generally built as a separate unit, and so can be considered apart from the main slab forms.

At the wall columns are generally, but not necessarily, half-depressed panels with brackets on the columns.

There are two main systems of building the forms, which can be called the "one-way" and the "two-way" systems. In the one-way system the joists are directly carried on the posts and in the two-way on ledgers. The latter, as seen in Chapter V, takes less timber than the former.

As regards labour cost, contractors usually have their individual preference, generally because they are more used to one system than the other. Some will claim that the one-way system is cheaper because the timbers run in only one direction and there is only one row to level up; and others claim that the two-way system is cheaper because there is less timber to handle. The two-way is the more common method.

In either method the floor boards are built into panels, and the main problem is to lay out these panels to the best advantage with as few different sizes as possible.

The joists may or may not be attached to the boards; usually they are not attached, the boards being held together with light battens about 3 ft. apart. Floor bays in this type of construction are made as

nearly square as possible ; about 20 ft. is the standard spacing of columns, though it may vary from 16 ft. to 25 ft. The side of the depressed panel is always about one-third of the column spacing, and the diameter of the column cap always about 0·225 the spacing of the columns.

A typical floor bay should be laid out on paper before deciding on the size of the panels, spacing of joists, posts, etc.

It is preferable to have joists, ledgers and posts spaced symmetrically in each bay, but this cannot always be done with posts and at the same time use them to the best advantage. Several different combinations of timber sizes could be used to carry the load, but it has been found from experience that the best sizes to use are timbers 3 in. to 4 in. wide and 4 in. to 6 in deep Deep planks are not advisable for joists or ledgers, as they are less stable and more liable to twist out of shape. Floor panels are always 1 in. stock, tongued and grooved

There will be the choice of using large sizes and panels with fewer pieces to handle, or small sizes and panels with correspondingly more separate pieces to handle. In general, the larger the building the larger can be the timbers and panels that can be used economically, as there will be more facilities for handling them both with labour and plant. The larger the sizes the faster will be the work of erection.

Floor panels and drop head panels are the only units made up in advance.

As with other types of construction, and perhaps more so with flat-slab floors, since short pieces cannot be so easily used up, the timber should be ordered from a sketch of a typical bay so that the lengths can be specified for the least waste. For instance, if the economical spacing of the posts is 6 ft., the ledgers should be 12 ft. long ; if they come on the job 10 ft. the posts must be 5 ft. on centres and 20 per cent. more posts will be required , or if they are 14 ft. long 15 per cent. of each ledger must be wasted. It is very easy to overrun the timber bill by 15 to 25 per cent. Also the forms should be built to a definite design so that the right sizes and lengths are used in the right place.

One-Way System (*Fig.* 75).—No ledgers are used, the joists being supported directly on the posts. The spacing of the joists is governed by the maximum allowable span of the sheathing (*Table* 1), and the spacing of the posts by the allowable span of the joists (*Table* 4). The spacings may have to be varied slightly for symmetrical spacing. The spacing of the joists should preferably be an even division of the spacing of the columns, placing a joist on each centre line. The sizes commonly used are 3 in. by 6 in. or 8 in.

Fig. 75 shows a typical lay-out for 20-ft. square bays. Laying out the floor panels symmetrically, there will be four panels 3 ft. 6 in. wide by 6 ft. 6 in. long and eight panels 3 ft. 3 in. wide by 10 ft. long. Panels usually run across the building and the joists lengthwise. Of course, several other combinations of panels can be used, but the end joints must be over a joist and it is preferable that the joints be on the centre line of the bay where the construction joints will be. It is usual

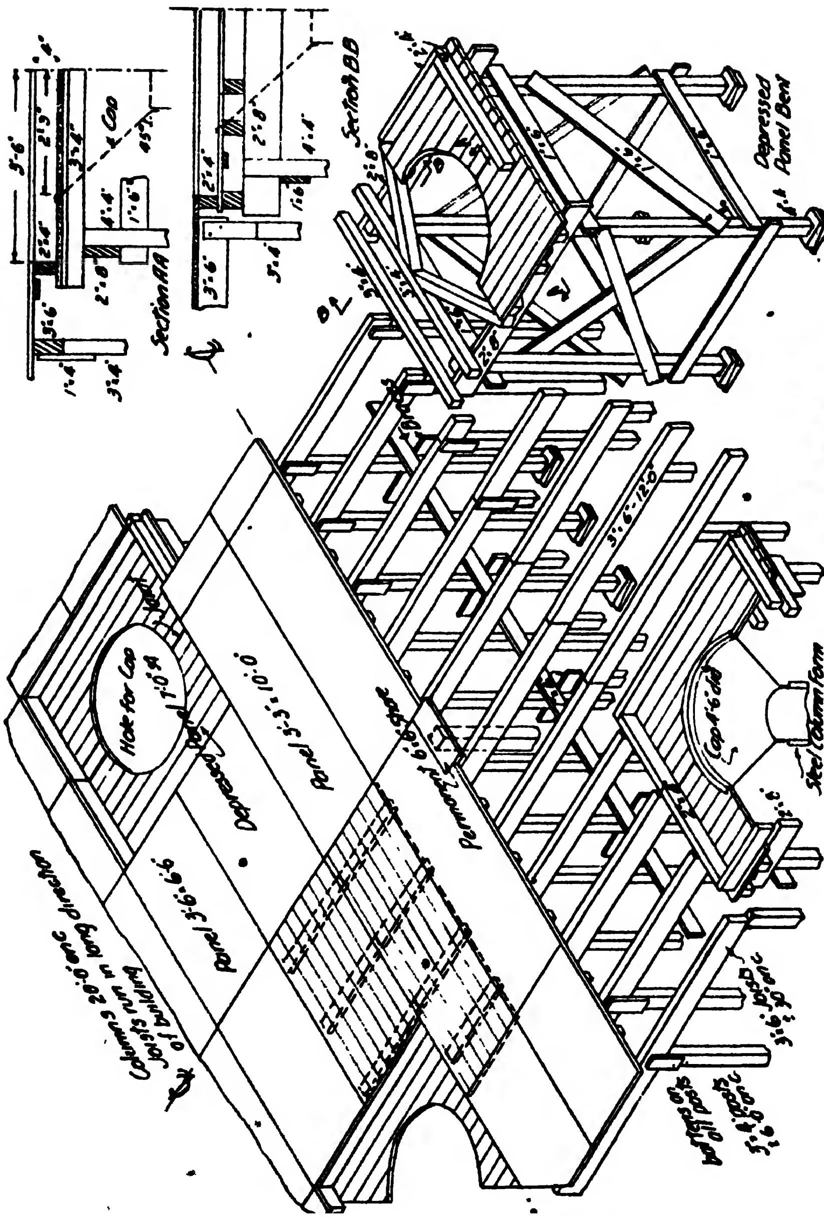


Fig 75 One-Way Design-Flat Slab Forms

to strip back to a construction joint, and if the panels overlap a joint they cannot be stripped. Posts must be set on wedges for adjustment and stripping and be cross-braced. Each post should be braced across the building, and every other post braced longitudinally. Since the posts are close together in this system and do not carry their maximum load, the bracing is less important than with the two-way design.

There should be sills under the posts when they set on the ground. Joists should be cleated to the posts, the cleat being permanently nailed to the post and temporarily to the joist.

In erection the posts are spaced and set first, with the wedges under them, and cross-braced. Using the braces as scaffolds the joists are next set and the posts wedged up to the required elevation. Last, the floor panels are laid loosely on the joists. At the wall beams the panels and joists are carried on the beam sides. The depressed panel forms are built separately as described later.

Two-Way System (*Fig.* 76).—In this construction the joists can be smaller and are carried by ledgers, which reduce the number of posts required.

Several different arrangements can be used, that shown in *Fig.* 76 being one of the commonest. A rectangular bay is shown, but it would be similar for a square bay. The panels can be made larger or smaller as desired, but should be as large as can be conveniently handled. The joists are usually 3 in. by 4 in and the ledgers 4 in. by 6 in. carried on 4 in. by 4 in. posts

The panels run across the building, as before. The joists butt or lap on the centre line of the bay and run in the longest direction. If the joists are lapped the panels in the centre of the bay will be staggered the width of the joist. There is one short line of ledgers between the depressed panels and three long continuous lines in the centre of the bays.

The bents, or posts and ledgers cleated together, are set up first on wedges, each post cross-braced in both directions. The joists are laid loosely on the ledgers and the posts wedged up to correct height; finally the panels are placed, also without nailing down. In the two designs shown (*Figs.* 75 and 76), the floor panels are nailed together with light battens and the joists are loose. *Figs.* 77 and 78 show photographs of forms built by the two-way system, in various stages of construction. Joists nailed to the sheathing in place of the battens can be used advantageously under certain conditions. Such conditions are shown in *Fig.* 79, where the bays are 21 ft. square and the depressed panel 7 ft. square, the ratio between the side of the depressed panel and that of the bay being such that all floor panels can be exactly the same size. For these conditions it is necessary that the bay and depressed panel be square and that the side of the depressed panel is one-third, two-fifths, three-sevenths or three-eighths of the side of the bay. The depressed panel can always be designed to meet these conditions; it is never less than one-third and seldom greater than two-fifths.

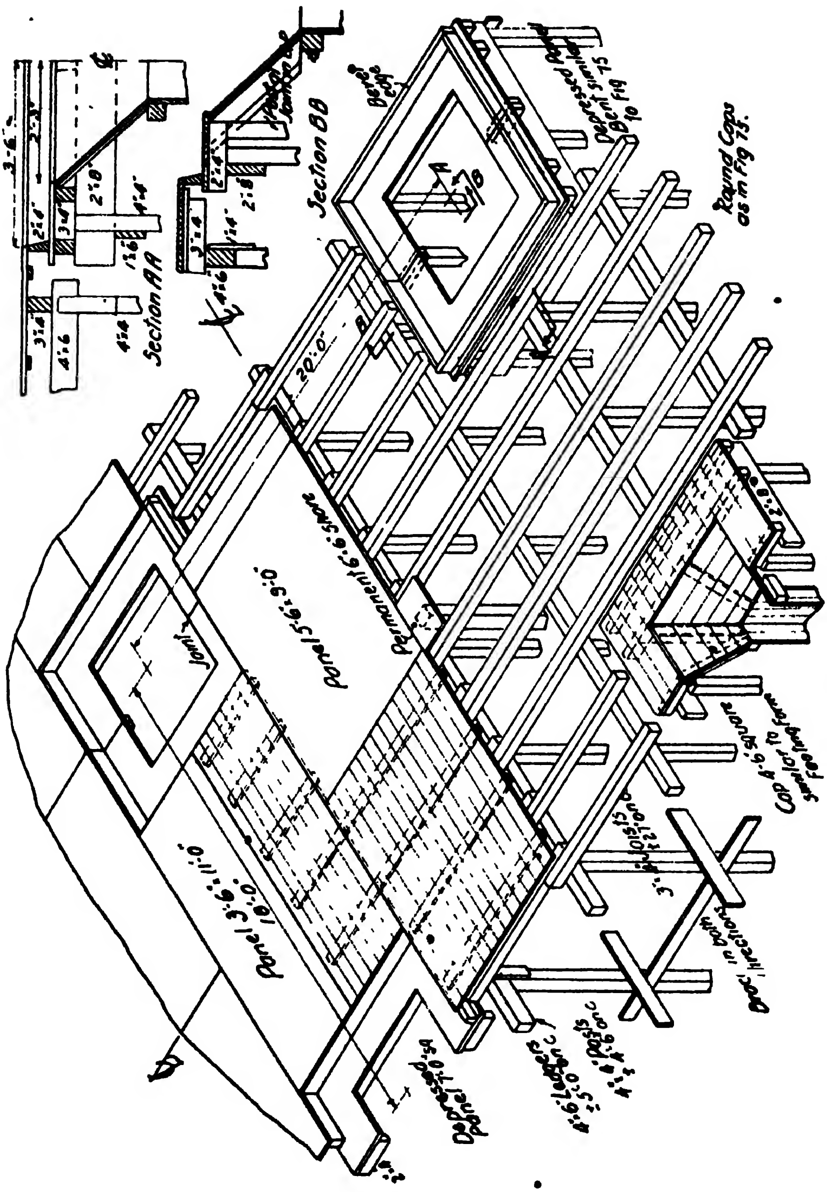


Fig 76 Two-Way Design-Flat Slab Forms

The panels are made up with a joist on each edge and one in the centre running in the long direction with the boards running across. The joists can be 2 in. by 4 in. or 2 in. by 6 in., depending on the span and

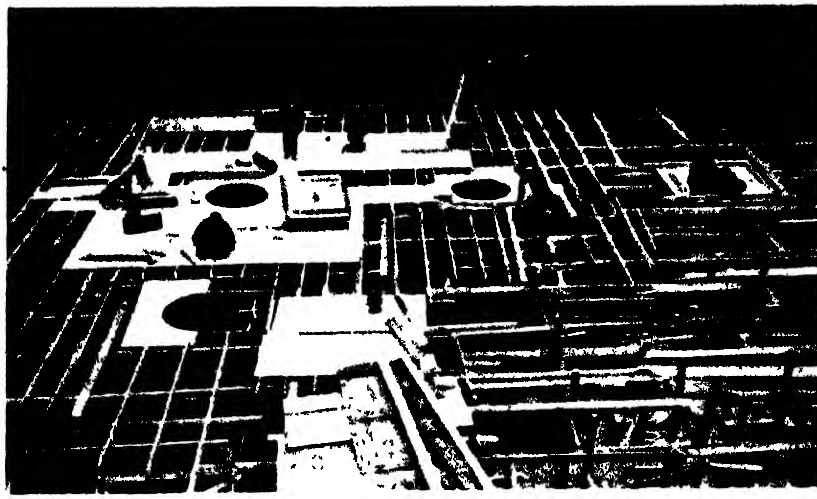


FIG. 77.—"FLAT SLAB" FORMS IN COURSE OF CONSTRUCTION.

FIG 78.—"FLAT SLAB" FORMS (LATER VIEW OF FORMS SHOWN IN FIG. 77).

slab thickness. Since all panels are the same size they are interchangeable and can be rapidly handled, thus speeding up the work. There is a disadvantage, however, in that the boards have to be cut up in short lengths, but this does not matter in a large building where the panels

can be used many times over as there would be little salvage however they were cut.

It will be noticed that in the centre of the bay there are three con-

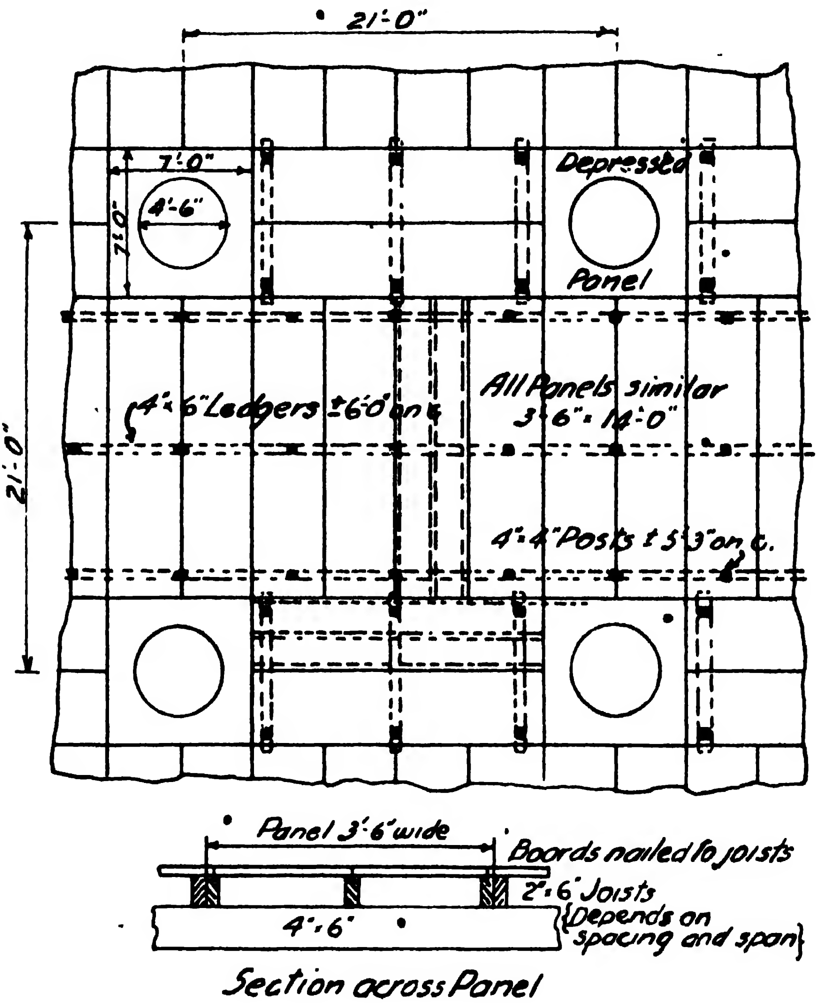


Fig. 79 Square Bays with all Panels similar
Side of D.P. = 1/3, 2/5, 3/7 or 3/8 Spacing of Cols.

tinuous rows of ledgers, as in *Fig.* 76, but that between the depressed panels there are three short rows running in the opposite direction. This is less convenient for stripping if the panels are to be used again on the same floor, or if part of the panels are required on the floor above before the whole floor can be stripped.

Stripping and Re-shoring.

It is common practice (though not good) to strip a bay entirely and re-shore with a post midway between each column and one in the centre of the bay. As in warm weather the forms are stripped in four to five days, removing the shores entirely from a bay is dangerous, as it is liable to cause excessive deflection because the slab is thin and the span long, and wedging back the shores may reverse the stress in the concrete and cause cracks in the top of the slab.

It is best to build in at least one permanent shore, which should be a 6-in. by 6-in., and which is not removed while stripping but is left in for about 28 days. Using one permanent shore, it is placed in the

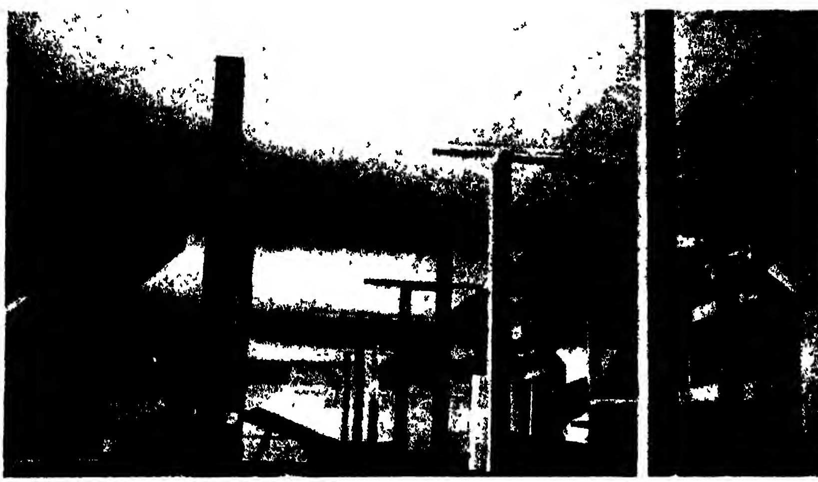


FIG. 80.—PERMANENT SHORES AND RE-SHORES.

centre of the bay and the slab is also re-shored with a post on the centre lines of the columns midway between the depressed panels. Re-shoring at these points is not so dangerous because there should be steel in the top of the slab.

The permanent shore consists of a 6-in. by 6-in. post on wedges with a board nailed on the top for a cap. The cap can be 12 in. square, or, better still, 12 in. wide and as long as the spacing of the joists. The cap is part of the floor sheathing and the panels are notched out around it (*Figs.* 75, 76, and 80). The shore thus has no connection with any other part of the forms, and these can be stripped independently.

When all five shores are built in permanently they can help to support the forms and do away with a corresponding number of posts. In this case the shores are made as above, but they have a 4-in. by 6-in. cleat nailed on one side to serve as a bracket to carry the ledger, which is set on wedges on top of the cleat so that it can be stripped without disturbing the shore (*Fig.* 81).

The centre line of the bracket, and not that of the shore, will be on the centre line of the shores.

It is important that the shores be placed vertically over each other in successive stories to prevent concentrated loads coming on the thin slab.

In a multiple-story building the two floors below the floor being poured should have all permanent or re-shores in place, and the story next below should have one permanent shore at the centre of each bay. The permanent shore at the centre of the bay should never be removed under 28 days, or longer in cold weather.

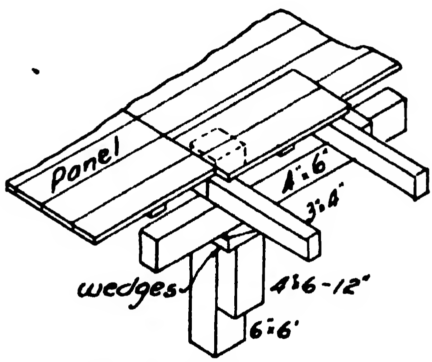


Fig 81. Permanent Shores.

The importance of re-shoring and permanent shores cannot be over-emphasised, especially in rapid construction. More failures are due to improper and insufficient shoring than to any other cause.

Depressed Panel or Drop-Head Form.

This is usually built as a separate unit; it is erected first and carries the floor panels. It can be constructed in several different ways, but generally consists of a four-post bent braced together in the form of a square with a ledger nailed on two opposite sides to carry the joists and sheathing. On top of the sheathing are the side forms to the depressed panel, generally made of 2 in. by 4 in.'s. A hole is cut in the sheathing to receive the cap form.

The sheathing is built up into one panel with 1-in. by 4-in. cleats, the hole for the cap is marked out and sawn, and the panel is then sawn through the centre, the joint being parallel to the direction of the boards; this is so that it can be stripped easily (*Fig.* 75).

Sometimes the joists are nailed to the sheathing, in which case they take the place of the cleats and are 2 in. or 3 in. by 4 in., depending on the span and weight of slab. In this case the joint is cut across the

sheathing (*Fig.* 76), and there is a cleat on each side of the joint and flush with the edges of the sheathing. The inside ends of this joint must be supported by posts.

Diagonal joists or cleats must be used to cut down the span of the sheathing when using round caps.

The four posts are set a little inside the corners of the depressed panel, and the two ledgers, generally 2 in. by 6 in. or 8 in., are nailed on the outside of the posts, the other two sides being braced at the top. The four-post bent should be cross-braced diagonally and horizontally braced at the bottom.

The side forms are often cut on a bevel, and they carry the main floor panels.

It is almost universal practice to use round steel column and cap forms, and since the depressed panel is erected before the cap the panel should be made a few inches larger in each direction than actually required to allow for adjustment and centering over the column. The hole in the panel should be cut slightly larger than the cap, as the cap form has to be slipped through it.

Although this is the commonest way of building the depressed panel form some contractors prefer to make use of the main floor joists instead of building the form independently. A method of doing this with the one-way design is shown in *Fig.* 82. Two rows of joists are placed a distance apart equal to the side of the depressed panel. These are set accurately and ahead of the other rows of joists. At each column a 2 in. by 2 in., or 2 in. by 4 in. if the joist is deep enough, is nailed on to the bottom of the joists at the depth of the drop. The depressed panel-form is made up as shown on 2-in. by 4-in. cleats or joists and in two halves, the outside joists being about 3 in. from the edge of the boards. The panel is made the exact size in one direction and a few inches longer in the other in order to carry the side form, or it can be made the exact size in both directions and the side form nailed to the ends of the main joists. When the panel is set it is shored by a post at each joint and one in each corner under 3-in. by 4-in. ledgers carrying the ends of the joists. It is best to use bevel strips in the corners to give neat lines.

Using this system the forms can be erected faster than with the separate bent system, but the work has to be done more accurately as there is no leeway for adjustment over the column.

To strip, the 2 in. by 2 in.'s are removed and the depressed panel form is then independent of the main forms, and being in two pieces is easily stripped. In any method the depressed panel is stripped before the main slab.

Column Caps.

All interior columns finish with a cone-shape cap, the sides making an angle of 45 degrees with the vertical. Being usually round, steel forms are used. The top of the form is turned over to form a lip which rests on the sheathing; the lower end is attached to the steel column

form. In this method the depressed panel has to be stripped before the cap. To avoid this, as sometimes the steel form is required first, another method is to set the lip flush with the top of the sheathing, resting on an angle-iron ring carried on blocks nailed to filler pieces on the underside of the sheathing. By removing these blocks the steel form can be

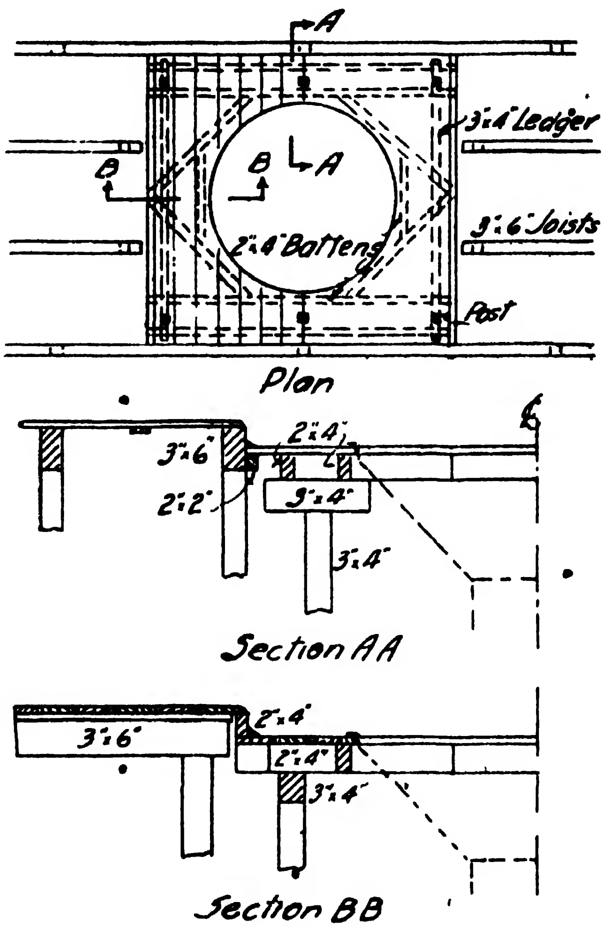


Fig. 82 Depressed Panel Form
Using main floor joists.

stripped independent of the depressed panel form. In another method the lip is replaced by an independent iron ring, which forms a neat joint, the top of the steel form being supported by blocks nailed to the joists (*Fig.* 83).

With square columns the caps will also be square, and they are generally made in wood, though steel forms for column and cap can be obtained.

They are generally only used when a square column and cap is desirable, as when it is necessary to attach cork insulation in cold-storage plants. The cap form is then really an inverted sloping side footing form, and is built similarly (*Figs*. 76 and 84, also *Fig*. 11, Chapter VI).

Other shapes, such as octagonal and ornamental moulded caps, can be obtained in steel.

The top diameter of the cap and the slope of the sides remain the same on all floors, so that the form has to be lengthened out at the bottom as the size of the column decreases.

Round Steel Column Forms.

It is convenient to consider round steel column forms here since they are nearly always a part of "flat slab" construction The moulds

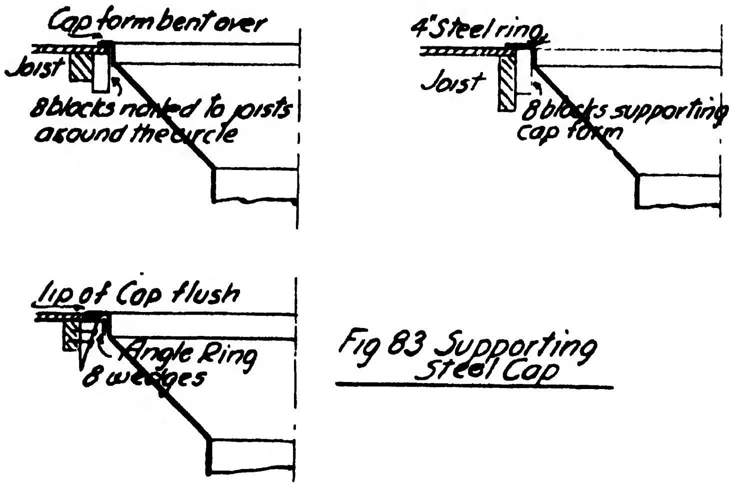


Fig 83 Supporting Steel Cap

may be bought outright but are more often hired for a particular job. The erection of them is often let to the firm from whom they are hired as they carry experienced erectors, though there is nothing complicated about their erection They are erected in sections, consisting of sheet metal with angle or plate stiffeners, and the sections are held together with steel wedges or clamps (*Fig*. 85 "*a*" and "*b*").

The moulds are adjustable to height and diameter, and as they can be stripped in three or four days it is only necessary to have sufficient forms for the lower story with the necessary adjusting pieces for the smaller columns above.

Wall Column Brackets.

Wall column brackets take the place of the interior column cap and are always used; they are shown on the left-hand side of *Fig*. 80. Their construction is similar to that shown in *Fig*. 22 (Chapter VII).

Construction without Depressed Panels.

When the depressed panel is omitted, the slab is made thicker. It is standard practice, but not quite so economical a construction unless the loads are light and the spans small. It is often used when the owner desires a perfectly flat ceiling without the projecting depressed panel.

Around the cap there will be a square panel in two halves as before, but it will be carried on the main joists and ledgers, the floor panels butting against it. The joists will run right through close to the edge of the cap, and when ledgers are used there will be one on each side the cap. Usually there will only be one joist that will be short, namely, the joist on the centre line of the columns.

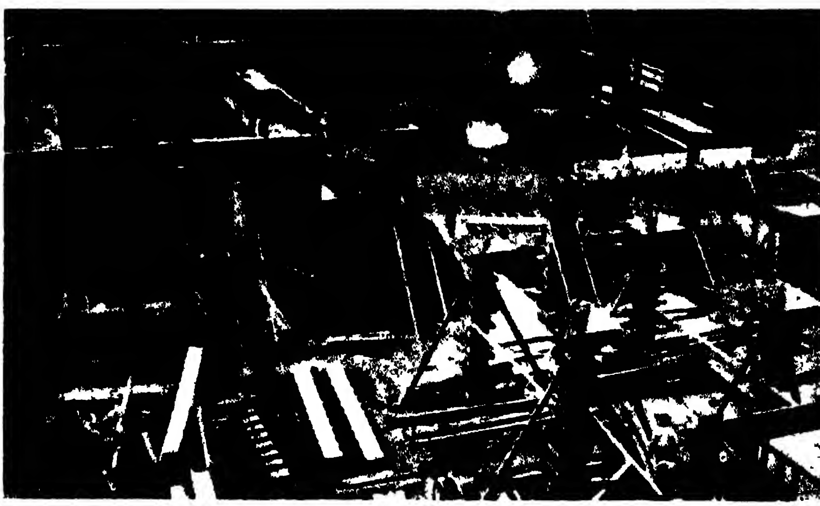


FIG. 84. SQUARE COLUMN CAP FORMS

Steel Forms.

In the past few years steel forms have been used extensively in flat slab construction by firms that specialise in this class of work. Whether they can be used economically or not depends on the area to be formed and the number of times they can be used.

If forms are to be used more than four times, the use of steel forms should be investigated. They are more often hired than bought outright, but if a firm is doing a large amount of this kind of work it will usually pay to buy them as they have large salvage value, low upkeep, and a long life, so that they can be used on several jobs.

Firms from whom the forms are bought provide working drawings of the forms, bill of timber, time schedules, and a man to superintend erection. This reduces some of the hazard in estimating the cost of the forms, as part of the cost will be definitely known.

FIG. 85a.—BLAW-KNOX STEEL COLUMN AND CAP FORM.

The shape of the building should be regular and the bays uniform with odd inches thrown into end bays.

The deck only will be steel, timber being used for the joists and

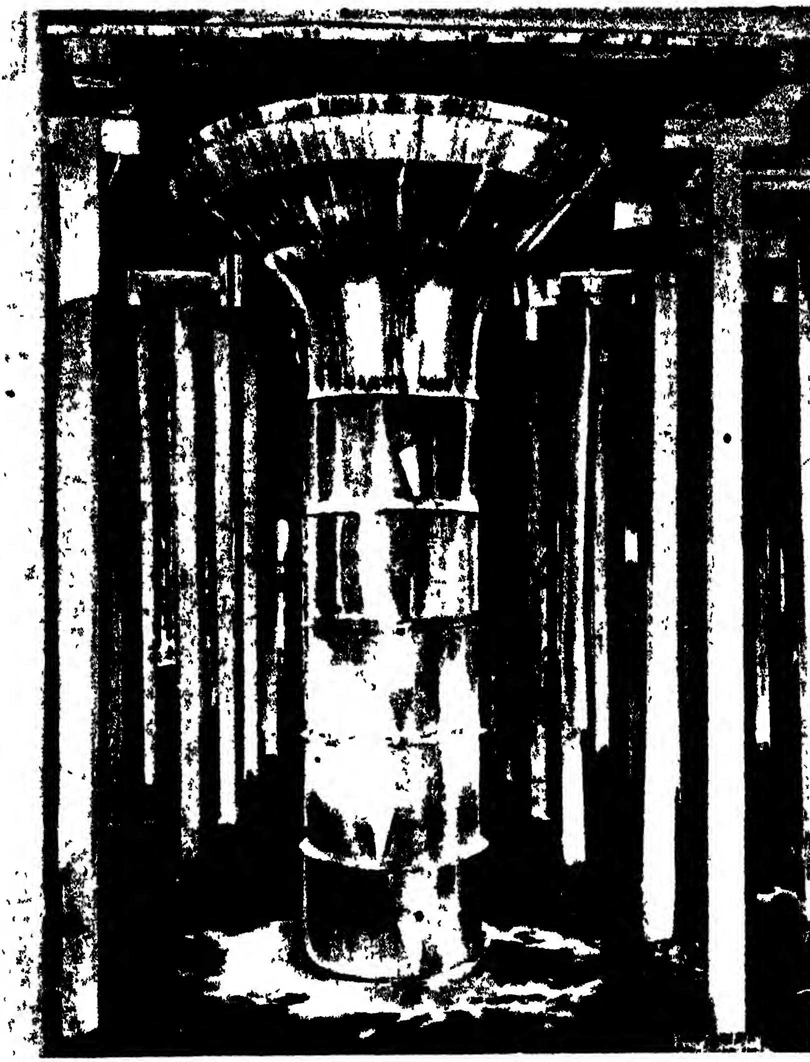


FIG. 85b.—DESLAURIERS STEEL COLUMN AND CAP FORMS.

posts. There is, however, a great saving in the timber required, about 50 per cent., since the joists are much farther apart and there are fewer posts.

Some of the advantages of steel panels are the greater speed of erection and the smaller number of workmen required, smoother finish

FIG. 86.—PERMANENT SHORES WITH STEEL DECK PANELS.

and hence less rubbing and pointing, less fire hazard and greater conductivity of heat, which is important in winter work. Another great advantage is that reshoring is unnecessary, as special shore panels are provided which remain in place while the other forms are stripped (*Fig.* 86).

The deck panels consist of pressed steel plates reinforced with steel channels and angles. They are made in a few standard sizes varying from 12 in. to 24 in. wide and 4 ft. to 8 ft. long.

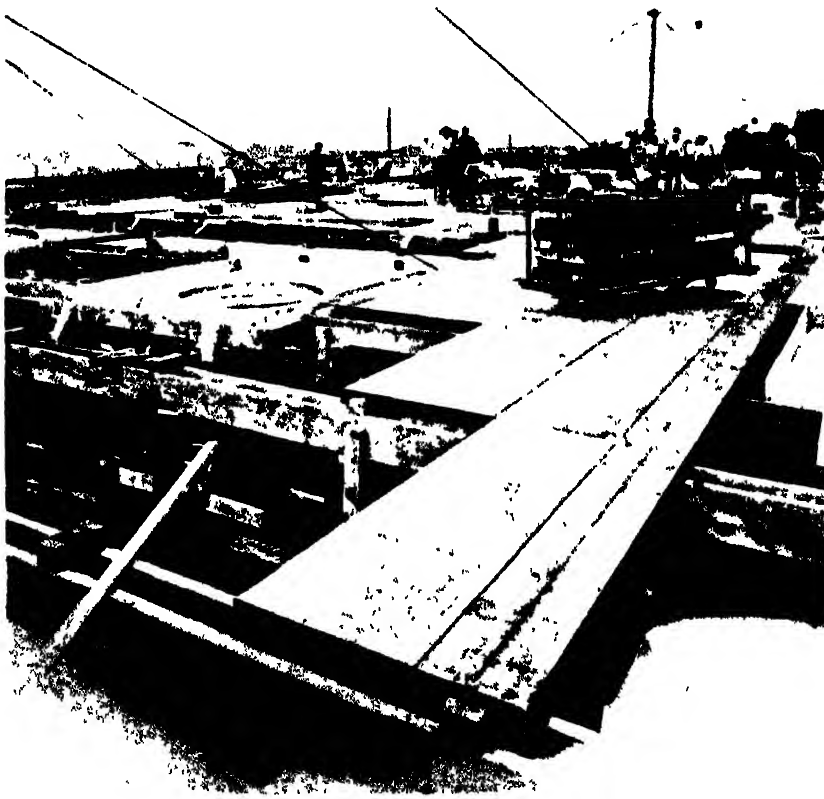


Fig. 87.—Blaw-Knox System of Steel Deck Forms

The one-way system is used, the steel panels being carried loosely on joists 4 to 6 ft. on centre, supported on 4 in. by 4 in. posts, 5 ft. to 6 ft. apart. It will be seen, therefore, that there is a great saving in the number of posts.

Drop panels can be formed entirely of timber or with steel decking; they are supported by either of the methods shown in *Figs.* 75 and 82. The four posts supporting the 3 in. by 4 in. ledgers in *Fig.* 82 may be omitted, and the ledgers hung from iron hangers from the main joists.

Fig. 87 shows the Blaw system and *Fig.* 88 the Deslauriers system of metal forms.

Estimating Cost.

It will be assumed that both types of construction will cost the same, since it depends on the amount of experience the contractor has had with one type or the other.

For buildings up to six stories in height it is usual to allow sufficient timber to form one complete floor or a floor-and-a-half, and for higher buildings to allow for two complete floors. The first floor will require ⅓ cu. ft. of timber per square foot of floor area, measuring the gross area, not deducting for depressed panels and column caps. To this amount

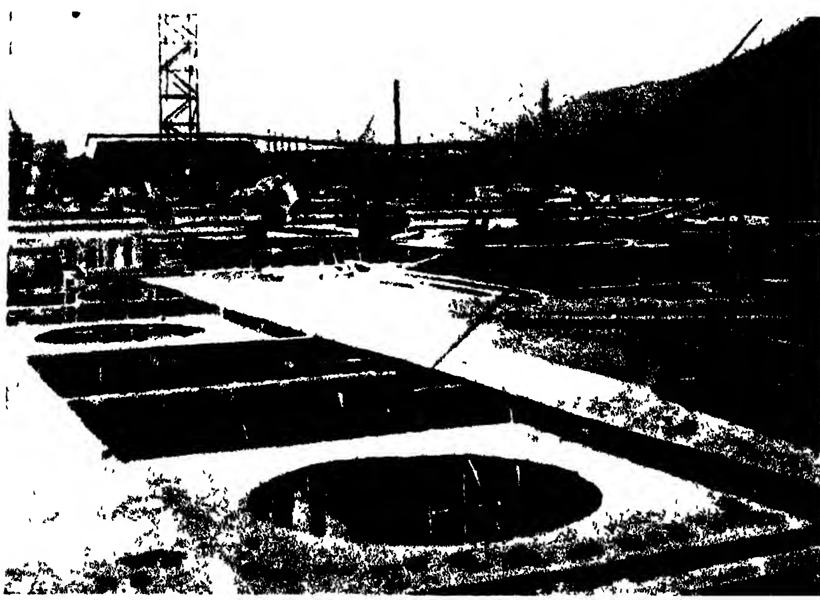


FIG. 88. DESLAURIERS SYSTEM OF STEEL DECK FORMS.

should be added 5 per cent. for each time the form is used after the first. If the one-way system is used about 10 per cent. should be added to this quantity.

The labour cost for the first floor will be about 20 per cent. higher than for the floors above, since there will be the panels to make up and the timbers to cut to the required length.

To frame, erect, and strip forms for 100 sq. ft. of the first floor will require about 10 hours carpenters' time and 6 hours labourers' time. For the floors above, including hoisting and cleaning, 100 sq. ft. will require about 6 hours carpenters' time and 8 hours labourers' time. Lowering the timber from the roof will require an additional 1 hour labourers' time.

Cost of 100 *sq. ft. forms for flat slab,* 1*st erection :*

10 hrs. carpenter at............................=
6 hrs. labourer at ....=
Timber = $1\frac{1}{3}''$ = $33\frac{1}{3}$ cu ft. at..........--

For each subsequent erection ·

6 hrs. carpenter at.... ....
8 hrs. labourer at..... ... .
Timber = 5 per cent of $33\frac{1}{3}$ $1\frac{2}{3}$ cu ft at.... ....

Cost of 100 *sq. ft. of forms used* 4 *times, calculated on whole area :*

7 hrs. carpenter at.. .. . .
$7\frac{3}{4}$ hrs. labourer at .. ....
Timber = $38\frac{1}{3}$ cu ft at . . .

The erection of steel column and cap forms can best be estimated by the lineal foot of column, measured from floor to floor, as the cost will not vary much with the size of the column The cap and column are taken together, and are not estimated separately.

An erector and helper together should erect and strip a 10-ft. to 12-ft. column and cap form in 4 to 5 hours

CHAPTER XIII.

FORMS FOR CONDUITS, SEWERS AND CULVERTS.

Conduits and Sewers.

THESE may be designed in many different shapes, and in size from 3 ft to about 12 ft diameter. The main types are shown in *Figs.* 89 to 94 which are all 7-ft. sewers or conduits The necessary changes in form design for variations of these shapes will easily be seen.

For the smaller sizes the circular shape is usually used Small circular monolithic sewers are seldom built now, however, as generally it will be more economical to use concrete pipe If there is a considerable length of conduit or sewer to construct the use of steel forms should always be investigated, as they will undoubtedly prove more economical Where concrete pipe and metal forms cannot be obtained, or are not economical, wood forms must be made up

There are several methods of building the forms, any one of which is adaptable to any shape to a large extent, though there is usually one best method to suit a particular shape

The differences in design are mainly in the methods of stripping or collapsing the forms so that they can be used repeatedly. This can be done in either of two ways. The form can be collapsed, without taking it apart, sufficient to clear the concrete, and then the whole form can be pulled through the sewer which has been poured. The other method is to take apart the rear sections, pass through the forms ahead and re-erect, bolting on the last section, and repeating the operation until the sewer is completed.

Usually sufficient forms are built for about 50 ft. of sewer, but on big work 100 ft. to 200 ft and even more may be necessary; the size of the job and the time allowed will govern this.

When the forms are collapsed and pulled through, they may be built in units 50 ft. to 100 ft long, depending on the size of the sewer and the methods used to pull them The pulling may be done by a team of horses, hoisting engine, windlass, winch, or by block-and-tackle, etc. Covering the wood sheathing with light sheet metal will lessen the friction and give a smoother finished surface. On smaller sewers the sheathing is sometimes covered with heavy building or tar paper, which enables the form to slip through the paper, while the paper is easily stripped off the concrete. If the forms are taken apart they should be built in sections longitudinally which are convenient to handle, say, 8 ft. to 12 ft. long.

Using the collapsing method without taking apart, all the forms are tied up until the whole section is poured and has set long enough to be stripped. This is a disadvantage when the invert and sides are poured together and when no outside forms are required, since while the concrete is setting no forms can be placed. If, however, there are outside forms and the invert can be poured ahead, these forms can be stripped and carried ahead while the concrete is setting If the forms are designed to take apart and sufficient forms are built for four or five days' pour, concreting can be continuous, as the rear section can be stripped and placed ahead while the last section is being poured

In general, smaller sewers are built with collapsible forms and larger ones with sectional forms designed to be taken apart in sections. The method used will largely govern the design of the forms. Another consideration is whether the invert can be poured ahead of the sides and top This is usually the case, and is more economical, as the invert forms and concrete can be carried ahead of the sides, saving time and giving a solid foundation for the side forms Sometimes, however, it is required that there shall be no longitudinal construction joints, in which case the whole form has to be made and set before concreting.

If the invert has a fairly flat curvature, it can be shaped with a template without using a form

Outside forms may or may not be necessary, depending on whether the trench can be excavated to the exact size without falling in When outside forms are required they will always be straight and generally vertical, and built and braced like ordinary wall forms. The outside of the crown of the sewer may be curved, but more often it has two straight sloping sides and a flat top While the latter takes a little more concrete, it saves the labour of building outside curved forms and using a template for the crown

The inside forms consist of sheathing nailed to curved ribs The sheathing may be 1 in. to 2 in thick (usually about 1½ in), the thicker sheathing will stand repeated usages better The width will be 4 in to 6 in., depending on the curvature If the curvature is sharp the sides of the sheathing boards will have to be bevelled.

The ribs are cut out of 2-in plank, for the larger sizes two thicknesses for each rib may be required The number of the ribs used and the curvature will determine the width of plank required, 8 in. to 12 in. being usual. The spacing of the ribs will be governed by the allowable span of the sheathing, which will depend on the height of the side walls and the thickness at the crown, usually the former governs. The ribs are spaced the same throughout the sides and top. The greater the spacing of the ribs the fewer joints there will be. At the points where the ribs butt or overlap, the depth of the ribs should not be less than about 6 in.

After determining on the type of the design from the general considerations already mentioned the sewer section should be laid out to full size on a platform of 1-in. boards cleated together. The ribs and

joints are then marked out and templates made. It must be remembered that the radius of the ribs will be the radius of the section less the thickness of the sheathing.

The method of connecting the ribs will depend mainly on whether the form is to be collapsed and pulled through as a whole, or taken apart. In the former case some or all of the joints are fixed, while in the latter the joints are made with bolts for easy taking apart.

The designs shown are to some extent interchangeable; that is, though shown for one particular shape they may be adapted to one of the other shapes. They can also be combined, using parts of different designs in the same form.

Figs. 89 and 90 show two methods of building collapsible forms. In *Fig.* 89 the joints are all made by butting the ribs and nailing cleats on each side. The side joints are cut on a bevel so that the sides can slide inwards. The cleats are nailed permanently to the sides and temporarily to the top and bottom ribs, with the heads of the nails left projecting, or bolts can be used. The sheathing must be bevelled at the same angle as the ribs.

A strut is wedged against the side ribs. To collapse, the side cleats are loosened, the wedges loosened, and a rod with hooked ends and a turnbuckle inserted in the holes in the side ribs to draw them inwards, sliding on the bottom ribs. After being pulled through the concrete the wedges are tightened and the cleats nailed on or bolted. As there will be friction over the whole of the invert, with this method it is best to cover the form with sheet metal or paper. This construction is best suited to the smaller size sewers.

If the invert cannot be poured ahead, the form must be blocked up from the foundation, preferably with concrete blocks. Sometimes a layer of concrete is placed in the trench and the form blocked up from this. The top side forms have the studs spaced the same as the side walls; the studs overlap and are nailed or bolted together, with braces across the top. Spacers are used to keep the inner form in position and to give the required thickness of concrete.

Fig. 90 shows a circular sewer form with the invert poured ahead. The ribs in this case are shown to overlap, with square ends. This saves some labour in cutting the ribs but does not give so strong a joint as using cleats and butting the ends. However, some flexibility of joint is necessary in this design. A brace at the top and bottom, and one in the centre if necessary, hold the form to shape, and a rod and turnbuckle are used to collapse the form after removing the braces. The form only has to be collapsed slightly for it to clear the concrete. The curved top form extends about half-way to the crown and is nailed to the side studs or braced back to the sides of the trench.

Fig. 91 shows a horseshoe-shape sewer or conduit with the invert poured ahead, being shaped with a template. This form is designed in five sections to take apart, and in longitudinal sections of 8 ft. to 12 ft. On the ends of each rib are nailed 2 in. by 6 in. as shown, through which

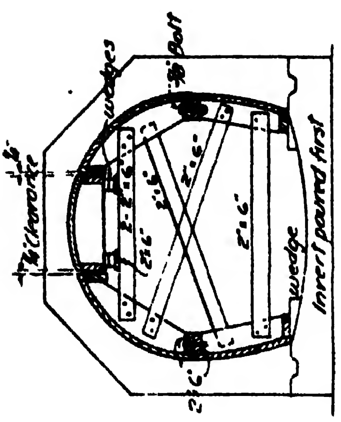


Fig 91. Horseshoe

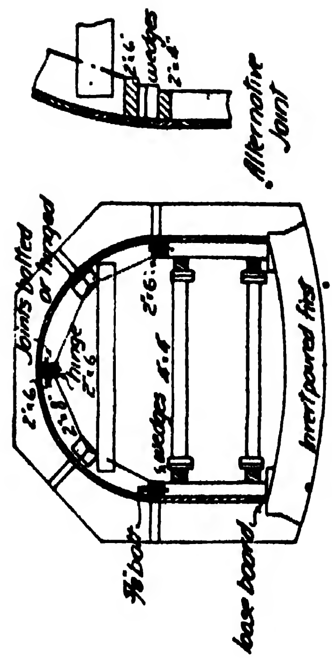


Fig 92.

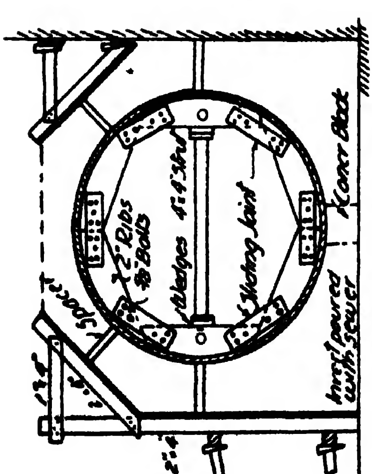


Fig 89 Circular Sewer
Ribs cleated, sliding joints

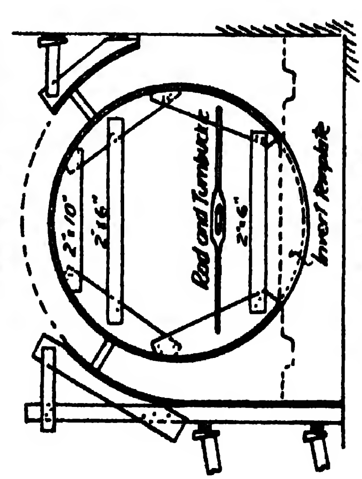


Fig 90. Circular Sewer
Ribs overlap. one piece form

pass the ⅝ in. bolts holding the two side sections together. The top section is made with about ¼ in. clearance at each end between the sides, and is supported on wedges resting on two 2 in. by 6 in. or larger cross-braces bolted to the side ribs. The lower ends of the sides are wedged up from the invert. Longitudinal braces should be nailed over the ribs in large sections in order to stiffen the form. A strip of heavy paper or sheet metal will cover the clearance openings if necessary.

To strip, the top section is lowered and the braces removed, leaving the sides free, and these can be unbolted or removed as a whole. This design is suitable for the largest size sewers.

Fig. 92 shows a semicircular sewer with straight sides and curved invert. The arch centre is a combination of the designs in *Figs.* 89 and 91. The joints at the haunches can be made similar to that at the crown if desired. For smaller sewers, instead of bolting at the crown they may be hinged, the hinges being fastened to the bottom of the vertical 2 in. by 6 in.'s.

The side forms are ordinary wall forms wedged from the invert, with a 2 in. by 6 in. nailed on top of the studs to give bearing to the arch centre to which it is bolted. Struts are wedged in between the walls.

The side forms are stripped first, after removing the wedges and bolts. To facilitate stripping the bottom board on each side form should be loose or lightly nailed, as this is where the form will tend to stick. It is not likely that the centre will fall when the sides are stripped so long as the braces are left on, but a few temporary shores may be advisable, and should always be used if the sides are stripped the day after pouring, in which case, too, the ends of the centre should be re-shored. This form is built in longitudinal sections of 8 ft. to 12 ft., bolted together through the studs and ribs, the sheathing being cut off flush at the ends.

If bolts are used at the crown, the centre is taken down in two pieces; while if hinged it is removed in one piece. To make it easier to remove the side forms, the centre may be set on wedges as shown in the alternative joint. In this case the top board on the side must project above the studs the depth of the wedges. Often in this design the walls are poured ahead and stripped before setting the centre, in which case it is set on posts and wedges along the walls.

Fig. 93 shows an egg-shape sewer, a very common form for the larger sizes. It is designed similarly to the form in *Fig.* 91 with the addition of an invert form in two sections bolted together. The invert is intended to be poured with the sides and top. As before, the top ribs are removed first. In this case 2 in.'s by 6 in. are nailed to the bottom of the top section to give bearing on the wedges.

Fig. 94 shows an ordinary box sewer or conduit, and the same design may be used for culverts. A centre ledger and posts will only be necessary in the larger sewers, depending on the span and thickness of slab. The tops of the side studs should be slightly bevelled for easy stripping, and the top boards on the sides should be loosely nailed.

Removing the top wales and cross-braces enables the sides to be

stripped ahead of the slab. If there is no centre ledger the slab form should be shored before doing this. Another method of supporting the slab form from the side forms is shown at "*a*," and the construction for large haunches is shown at "*b*."

The forms for any other shaped sewer or conduit can be made by one of these methods or by combinations of methods. When pouring, the concrete should be brought up at the same rate at each side of the crown to avoid unequal loading of the form, and hence possible distortion and throwing out of line.

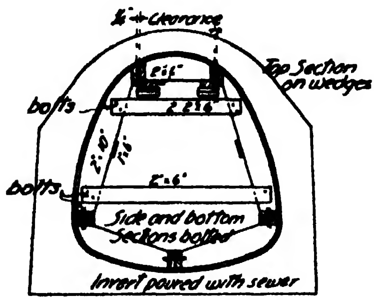


Fig. 93 Egg-shape Sewer
Top section stripped first

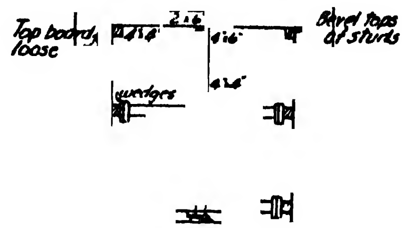


Fig. 94 Box Sewer
Sides stripped first

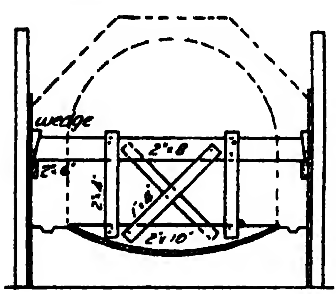
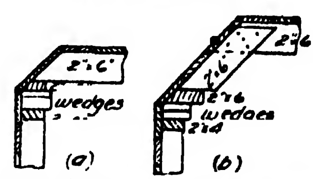


Fig 95 Supporting Invert Form



Slab form supported on wedges

Inverts can be stripped the day after pouring, side walls in one to three days, and arches in three to ten days, depending on the span. Flat slabs can be stripped in five to twenty days, depending on the weight and span.

Fig. 95 shows a method of supporting the invert form when it is poured ahead. The main thought to bear in mind when designing sewer and conduit forms is to provide for continuous operation of placing forms and concrete.

Culverts.

Culverts are of two designs, the arch or box. Either type may be open or closed, depending on whether there is an invert or not. Inverts

will require no forms, any slight curvature there may be being made with a template.

Box culverts are generally used for spans up to 8 ft. If the span exceeds twice the depth it is more economical to build a double culvert with a wall down the centre; in this case the forms can often be advantageously used twice. Arch culverts are in general more economical for spans greater than 8 ft. Above about 15 ft. span they can be classed as bridges, which will be treated in a later chapter.

The forms for culverts are usually erected complete, as the structures are too small to make it economical to build them in sections. Although some timber would be saved in doing so, this saving would be more than offset by the time lost, since the slabs are generally thick and so require

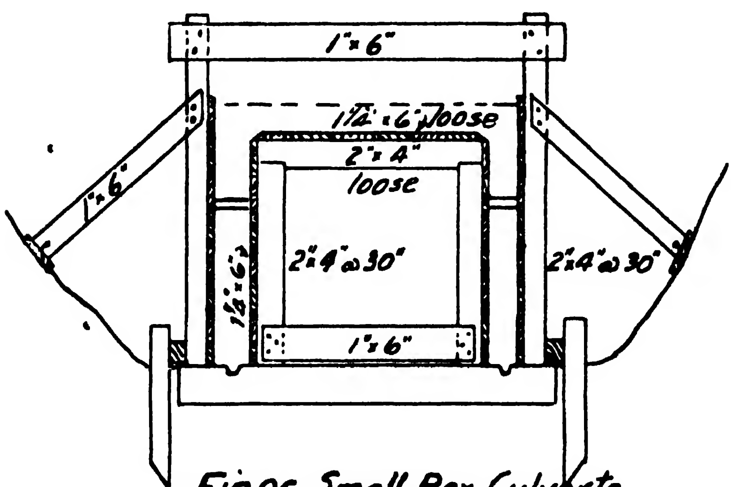


Fig. 96. Small Box Culverts

shoring for a considerable time before they can be stripped and the forms used again.

If the culvert is long, say, 80 ft. to 100 ft., it may be economical and save time to build the wall forms for one-half the culvert and use them twice; then, while they are being poured in the second half, the slab or arch form can be erected in the first half. In this case a full set of slab or arch forms is used, though with short-span arches it is possible to use the centres twice, as they can be stripped early.

Not much timber is saved by using wall forms twice, since there must be a row of posts along the walls to carry the slab or arch forms, so that the main saving is in the wall sheathing, which is usually not a large item. In general it will be cheaper to build a whole set of forms. This being the case, there is not the problem of building the forms in sections for easy stripping and re-use as there is in sewer and conduit construction.

In the case, however, of a box-culvert so small that it would be difficult for a man to work in the space to do the stripping, the question

of easy stripping becomes important. As large a free opening as possible must be left, and there should be few nails to pull or wedges to strike. *Fig.* 96 shows a method of building forms for a small box-culvert. The outside form is an ordinary wall form ; a few diagonal braces back to the sides of the trench are advisable to hold the form in line. The inside side walls are built in sections about 8 ft. long, butted against each other with the sheathing nailed to the studs. The slab form consists of loose boards laid on the joists, which are notched over but not nailed to the studs. A nail here and there will hold the boards in position. The bottom brace is either nailed to the studs or placed between them, in which case they should be cut a little longer than the clear span and wedged into place.

To strip, the joists and braces are knocked out and the side sections

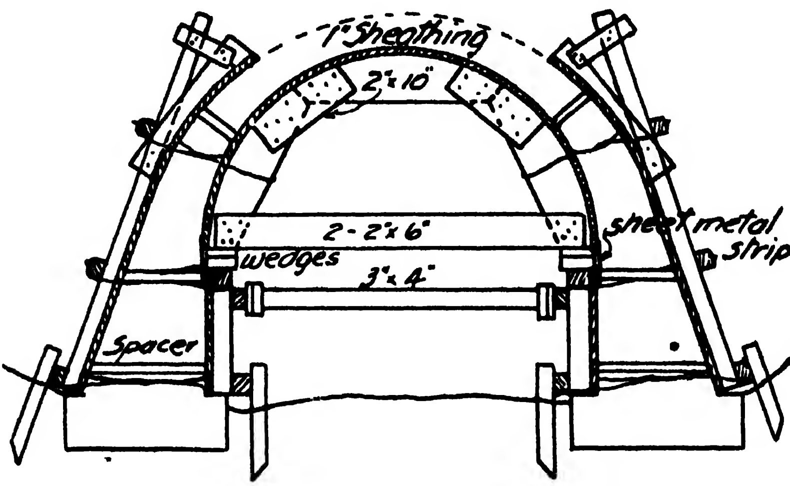


Fig. 97. Small Arch Culverts

pulled through the culvert. If the wall form is made in one piece it will either have to be pulled out as a whole or be taken apart inside the culvert, either of which will be difficult.

Forms for larger box-culverts require no particular mention, as they are straight wall and slab forms. The method of supporting the joists on the wall form is shown at " *a* " in *Fig.* 94. Wedges should preferably be over studs. The size and spacing of the joists will depend on the span and weight of the slab (*see Tables* 1 and 4).

There are several ways of building centres for arch culverts. The governing features will be the foundation conditions, presence of water, span, and rise of the arch. The stream can be diverted around the culvert, or carried in a flume through the culvert, or it may be necessary to confine it within cofferdams to enable the walls to be built in the dry.

There are three main types of design, illustrated in *Figs.* 97 to 99, differing in the methods of supporting the ribs.

For small spans, up to say 8 ft. or 10 ft. (*Fig.* 97), the ribs need only be supported at the walls, with a tie across at the ends. The centres are set on wedges on top of the wall-form caps. The main requirement of this type of centre is stiffness, so the joints in the ribs, which should be butt joints with cleats, should be securely nailed. The cleats should preferably be also cut to the radius. Sheet-metal strips nailed to the wall forms will close the gaps at the wedges, or loose boards can be used.

For longer spans this type of centre would become distorted, as each joint would tend to act as a hinge, so intermediate shores must be used, either one or two at the crown section and one at each haunch.

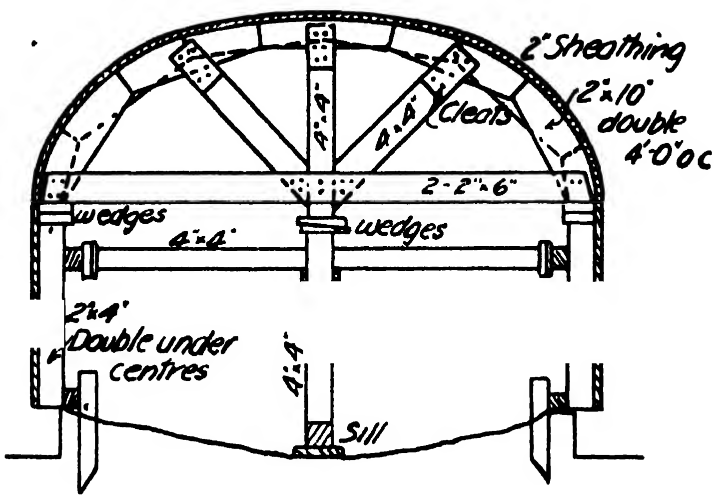


Fig 98 Large Arch Culverts
– Centre Post

There are two methods of carrying the shores to the foundation, namely, either concentrating them on a single centre post, or taking each shore directly to a centre footing.

Fig. 98 shows the former method. The three 4 in. by 4 in.'s supporting the crown and haunches with the two 2 in. by 6 in.'s form practically a truss, which is supported by a central 4 in. by 4 in. post with wedges at the top for adjustment and striking of the centre. The diagonal members should be bevelled at the top for even bearing on the ribs, to which they are cleated on both sides, and at the bottom they are nailed to the centre post and the horizontal tie.

A variation of this design, though not quite so good, would be to make the horizontal tie a 4 in. by 4 in. cleated to the ends of the ribs and to replace the 4 in. by 4 in.'s by two 2 in. by 4 in. or 6 in., one on each side of the ribs. The centre post should set on a sill, sufficiently large to reduce the pressure on the foundation to the required amount. A continuous

sill is better than individual sills under each post. The wall studs carrying the centres should be doubled, or a 4 in. by 4 in. used. This is perhaps the commonest type of arch centre for culverts from 8 ft. to 15 ft. span. The trusses can be built up complete, set in place and sheathed over.

In *Fig.* 99 the ribs are built in the same way as in *Fig.* 98, but all the supports are carried to a central footing. The posts are set as nearly at right angles to the ribs as possible, and the wedges set on top of the posts. The two 2 in.'s by 6 in. tie all the posts and the ends of the ribs together.

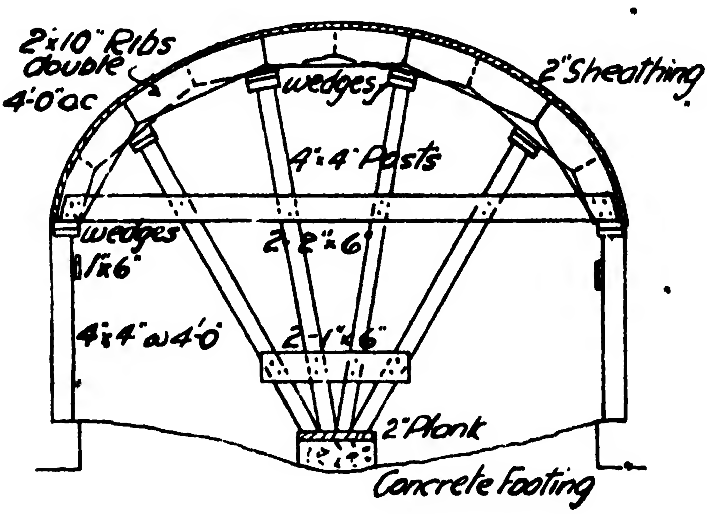


Fig. 99 Large Arch Culverts
Multiple Posts

This method can be erected more quickly than that shown in *Fig.* 98, and while it will take a little more timber there will be greater salvage as the 4 in. by 4 in.'s will be in long lengths.

When in doubt as to the bearing value of the foundation a continuous concrete footing is advisable, this is also useful to level up an uneven rock foundation. The concrete can be poured into a form set in the water if necessary, using fairly dry concrete and preventing the stream flowing through the form by putting some puddle around it. If the foundation is good the posts can be set on timber sills, built up to give the required area. Methods of building sills are dealt with in the chapter on Bridge Formwork.

If the stream is confined to the centre of the culvert it may be impossible to use a centre support, so a design such as shown in *Fig.* 100 may be used. The lower ends of the diagonal posts should be toe-nailed to the horizontal strut, which is cleated to the ends of the ribs. The pair of braces prevent overturning of the short vertical posts.

It rarely happens that there is so much water in the culvert that a truss design is necessary, using no intermediate supports, but if this is the case the design in either *Fig.* 98 or *Fig.* 100 can be adapted to truss form by making the connections of the members more rigid. In a true truss, if the centre is loaded symmetrically, there is little stress in the diagonal members and their main purpose is to add stiffness to the centre and prevent excessive deflection. Distortion and deflection are the main things to avoid, and this can be done by rigid joints and careful pouring of the concrete.

Since culvert forms are seldom used more than once the connections are made with nails instead of bolts, generally using 20*d.* nails.

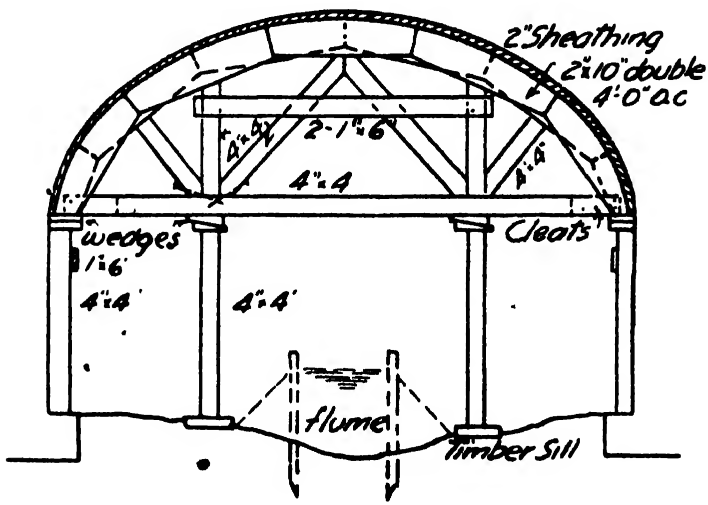


Fig. 100 Large Arch Culverts
Two Posts

It is usual to give a slight camber to the arch to allow for settlement and take-up at the bearing points, an ⅛ in. to ¼ in. will be sufficient for culverts.

Ribs can be either single or double 2 in. plank, 8 in. to 12 in. deep at the centre and about 6 in. deep at the ends. The wider the plank the fewer joints will be necessary. They should be cut from a template made by laying out the arch to full size.

Single ribs (*Fig.* 97) will be spaced from 24 in. to 30 in. apart, depending on the allowable span of the sheathing for the thickness of the slab at the crown. With single ribs 1 in. sheathing is used.

Double ribs are formed by nailing two thicknesses of plank together, breaking joints at the centre of each piece (*Fig.* 98). One-and-a-half to 2 in. sheathing is used with double ribs, and the spacing will be 30 in. to 48 in. There will be less shoring to do with double ribs, but if the curves

are sharp the edges of the sheathing boards will probably have to be bevelled.

The size of the rib planks need not be calculated, as 2 in. by 10 in. will always be heavy enough ; in fact plank 1½ in. thick will be satisfactory.

Posts and braces are commonly made too heavy. The loads and pressures are seldom so great that heavier posts and braces than 4 in. by 4 in. and 2 in. by 6 in. are required. It is more important to place them correctly, where they will do the most good, tying all parts of the form together to give maximum stiffness.

Abutments are usually poured first to the springing of the arch and allowed to set overnight before pouring the arch. The arch is generally poured from the springing on each side to the crown, being careful to bring the concrete up at the same rate on each side. This often causes trouble, as the concrete tends to push the form inwards at the haunches and up at the crown. This must be watched, and the wedges gone over and tightened up all the time while concreting and it may be necessary to pour some concrete at the crown to weigh down the form.

If wide planks cannot be obtained for the ribs, or if they are high in price, a 2 in by 10 in can be made by nailing a 2 in. by 4 in. on the edge of a 2 in. by 6 in., the former only being cut, giving full salvage of the latter.

Wedges should be 6 in wide, 8 in to 12 in long, and tapering from 1½ in. to ½ in.

Arches up to 6 ft. or 8 ft. span can be stripped in 5 to 10 days ; from 8 ft. to 15 ft. span in 10 to 15 days. The time must be judged from existing climatic conditions.

Estimating Cost.

The building of forms for reinforced concrete buildings has practically become standardised, and there is sufficient of this work to enable an estimator to compare unit costs and to obtain checks on his own method of estimating. In previous chapters the method of estimating the various units of formwork in building construction has been given in some detail, because with standard methods of construction the costs also should be more or less standard for a given labour rate. For most of the structures described in this and succeeding chapters it is, however, impossible to give any figures for estimating that could be used safely under all conditions without the possibility of serious error, so that data for cost estimating will only be given where experience has proved that the figures will not vary greatly with the conditions.

Sewers, conduits, and culverts, especially the former, are structures that vary considerably in design, in the methods of building, and in the conditions under which they are built, so that unit costs may have a wide variation. Size of structure also has considerable influence on the cost of formwork. In large structures there is time to make changes, improve methods, and train workmen that will often cut the labour cost in two.

The first cost of a sectional form can generally be estimated fairly closely, remembering that the quality of the work should in general be

better than in ordinary building forms, since the forms must be built strongly to withstand repeated usage and fit together accurately for easy assembly and stripping.

The cost of moving the forms will depend on the size of the structure, the method of collapsing, and whether they are moved by hand or power. A close study of actual conditions will often suggest some simple device that will cut down the cost of this part of the work.

Small culverts can be compared with similar forms in buildings, increasing the unit cost according to the local conditions under which the work has to be built, these are generally less favourable than in buildings.

The larger culverts are more similar to bridge construction and can be estimated similarly, as given in a later chapter.

CHAPTER XIV.

FORMS FOR TANKS, SILOS, BINS, STANDPIPES.

ALL structures having high thin walls, whether they are for the purpose of storing water, silage, grain, cement, sand, stone, etc., are built by similar methods. The same general principles apply whether the structure is square, rectangular or round, or whether it consists of one unit or a number of similar units connected together.

If the walls are square the forms are ordinary wall forms and so require no particular mention, and they are raised as described in Chapter VIII or in the same manner as will be described for circular forms. *Fig.* 101 shows the outside wall forms for square grain storage bins; the forms are one story high and raised a story at a time.

Since most structures of this class are circular, only circular forms will be described in detail, though the same methods can be used for square and rectangular structures. There are three methods of building wooden circular forms for high thin walls, namely, the fixed form, the so-called "moving" form, and the continuously sliding form.

If the tank or bin is not over 10 ft. or 12 ft. high and the diameter is not over 30 ft., it is more usual to build forms for the whole structure rather than to use moving forms. In this case the walls will be built with ordinary wall forms, straight or curved, as described in Chapter VIII.

Very high structures, such as silos, bins, standpipes and chimneys, or lower structures with large diameter, such as oil storage tanks, are built with moving forms or continuously sliding forms in order to save material.

If it is desired to have no construction joints, as in water tanks and standpipes, so that there will be less possibility of leakage, continuously sliding forms are used.

If the type of structure is practically standard, as with silos, or if the size of the job warrants it, as with a battery of storage bins, steel forms are the best to use.

While any form that is moved up the structure in successive lifts is commonly called a "moving" form, it must not be confused with a "continuous sliding" form, which slides up the concrete continuously without stopping after once being placed, whereas a moving form is stationary while being filled with concrete. Whether to use one or the other is a question of judgment, depending mainly on the size of the structure and whether joints are allowed or not. The cost of steel forms should be investigated in any case.

Where the yardage of concrete is small a continuously sliding form is

not suitable, as it cannot be raised as fast as the concrete sets and hence its extra cost is not warranted. When the yardage is such that a form 4 ft. to 6 ft. high will require twenty-four hours to pour, using a small-size mixer, then continuous sliding forms can be used to advantage if desired, though the structure should be fairly high or the extra cost of the forms and jacks will be more than the time and labour saved. With the use of rapid-hardening Portland cement continuous sliding forms may be used for many structures that are now built with moving forms.

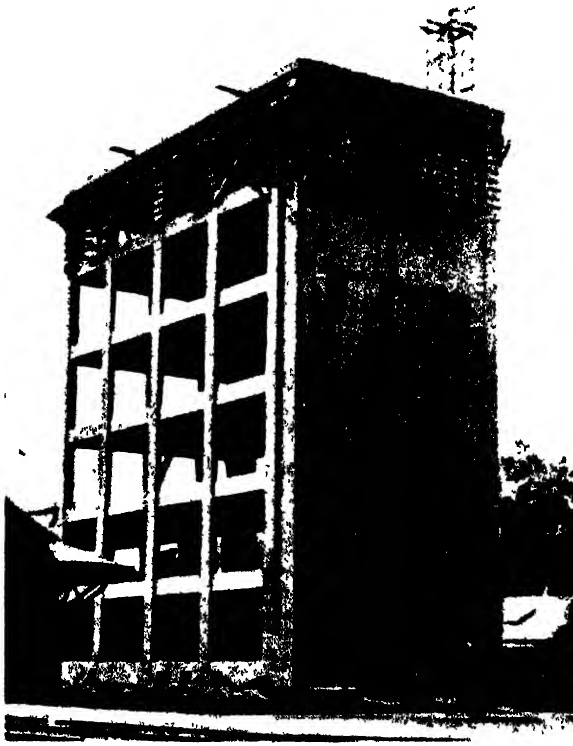


FIG. 101 – MOVING WALL FORMS 1 STORY HIGH FOR SQUARE GRAIN BINS.

Moving Forms.

Moving forms can be used whether the structure is above or below ground. If the diameter is not too large, structures below ground can be built economically by building a ring on the surface, excavating inside with a clamshell, sinking the ring by its own weight, adding another ring, continuing excavating, and so on until the required depth is reached, the form always being above ground. The bottom of the first ring should be bevelled to give a cutting edge.

The commonest structures for which moving forms are used are silos, though other structures may be built in the same manner. A description

of a silo form will answer also for any kind of storage bin or water tank, if the latter can be poured with joints. Although a jointless structure assures greater safety against leakage of water, many large standpipes have been successfully built with moving forms.

The height of the form should be 2 ft. to 4 ft., seldom greater; and if galvanised sheet metal is used for sheathing it will be governed by the commercial width of the sheets.

There are three common methods of raising the forms, either on a centre post, on several posts placed around the wall, or by using two lifts of forms and raising one on top of the other. The first method can be used for only comparatively small structures, the second is the usual way of building silos, and the third is used for large storage bins.

Fig. 102 shows a form raised on a 6 in. by 6 in. centre post. This method should not be used for diameters greater than 12 ft. or 14 ft. The inside and outside forms are built in four sections, each a quarter of a circle. To obtain the lengths of the inner and outer ribs the half-circles should be laid out full-size on a level platform. First the framework of a section is built up by nailing a 2 in. by 4 in. upright about 4 ft. long between each pair of ribs and a 3 in. by 4 in. at one end of a section. On this framework is then nailed 20 to 24 gauge galvanised iron, using 6*d*. nails.

The four sections are then assembled on the foundation, the outside sections first, and bolted together with ½ in. bolts. To the uprights of two opposite joints are bolted two 2 in. by 8 in.'s just above the top of the form and long enough to reach the outer upright, and two similar 2 in. by 8 in.'s to the lower ends of the inside uprights. These members carry the forms.

Between these 2 in. by 8 in., and at the exact centre, is a 6 in. by 6 in. post up which the forms slide. On each side of the post is placed a 2 in. by 6 in. about 5 ft. long, nailed to each 2 in. by 8 in., to form guides to keep the form vertical. At the back of the guides, cleats are nailed top and bottom. Each pair of intermediate uprights is braced back to the 2 in. by 8 in.'s both at the top and the bottom. Short spreaders are nailed to the tops of the uprights to stiffen the form and take the outward pressure of the concrete at the bottom. About two-thirds of the radius from the centre holes are bored through the 2 in. by 8 in., through which is passed a loop of rope, or preferably wire cable, and at the back of each hole is nailed a spacer.

Various means may be used for raising the forms. In *Fig.* 102 two hand winches are shown bolted to the post, or they can be fastened to the foot block. The winches must be equipped with a clutch so that the form can be stopped and held at any point. The winches are used with block and falls fastened to the top of the post.

After the form has been poured on the foundation, the following day the form is hoisted up until the bottom of the form overlaps the concrete about 4 in., when the clutches are locked and the form again filled. Each day the form is raised its own height.

Two large timbers should be bolted to the bottom of the post to

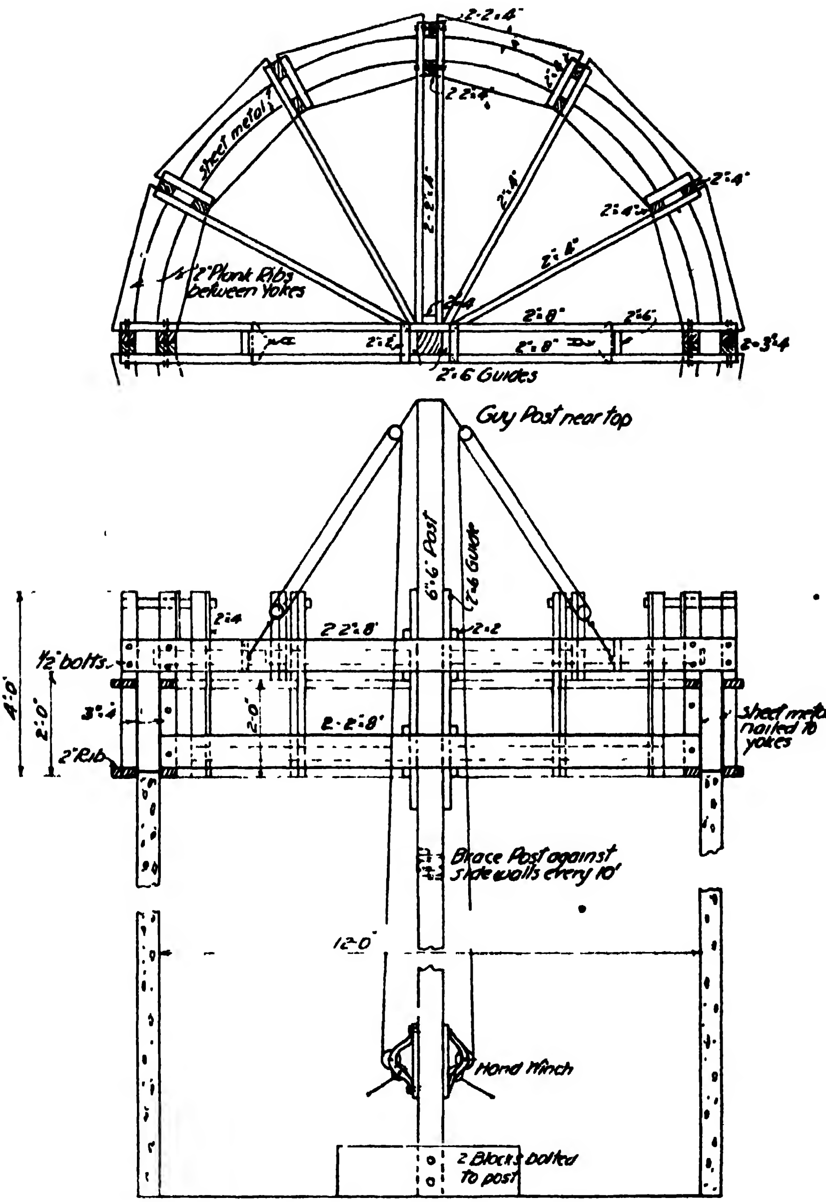


Fig 102 Moving Forms for Small Silo

distribute the pressure. The form being only 2 ft. high and the sheathing being metal there is not much friction against the lifting of the form. It is essential that the form be level before starting, and that the winches are operated at the same rate. It is possible to carry both cables to the same winch, but this is liable to cause trouble and it is better to use two.

If a post long enough cannot be obtained the splice must be carefully made and cleats nailed over the joint after the form has passed. The top of the post must be guyed, and as the form moves up the post should be braced to the sidewall in each direction every 10 ft. in height. The guys can be removed when they interfere with the forms.

For larger diameter bins the forms become heavier, and it is necessary to raise them in sections on several posts.

The design most used for silo construction is that known in the United States as the "University of Wisconsin Method" This is shown in *Fig.* 103. First lay out a cross section of half the silo to full size on a level platform, using a marker consisting of a 1 in by 2 in. lath nailed to a stake at the centre of the circle and with two holes near the end for inserting a pencil at distances from the nail equal to the inside and outside radii (if wood sheathing is used the thickness must be deducted and added respectively) Then mark off the ribs, using two to a quarter-circle, or more if necessary, so that the ribs can be cut from 12 in-wide planks. The minimum depth of the inner rib at *A* should be 6 in., and of the outer rib at *B* 4 in. The ends of the rib are cut along the radii, so that they will butt together. Where the wedges occur, on one side the upper and lower ribs will be cut back 2 in, and on the other side the upper rib will be cut back 2 in.

Having cut out sufficient ribs for two complete inner and outer circles, each pair of ribs is joined together by uprights. The outside ribs have a 2 in. by 8 in at each end and two 2 in by 4 in.'s between, or more if necessary, so that they are not more than 2 ft apart. The inside ribs have a 2 in. by 6 in. at each end and two 2 in. by 4 in.'s between, the latter mortised into the ribs. At the centre of each inside rib is cut a 4 in. by 4 in. hole, care being taken that the holes are vertically over one another. Two holes for ½ in. bolts are bored in every end upright, except next to the wedges.

The inner form is then assembled in place and bolted together, leaving a 4 in. space at the top at each quarter-point. Over each joint, top and bottom, is placed a 2 in. by 6 in. about 2 ft. long, and holes are bored through for ½ in. bolts. When bolted up and squared, 18-gauge galvanised iron is nailed on with *6d.* nails. If one sheet is not long enough for two ribs, two sheets should lap 6 in. at the joint.

Each quarter section of the outside form is assembled, the galvanised iron nailed on, and the form placed the width of the wall away from the inner form and bolted up. Spacers are used every 4 ft. or 5 ft. to maintain an even wall thickness. Wedges are cut to fit, as shown, after the form is assembled, and when wedged up the form is ready to pour after being levelled and plumbed.

After the concrete has set a 4 in. by 4 in., made up of two pieces of 2 in. by 4 in. nailed together, is placed through the holes of the inner ribs, having square bearing on the concrete floor. It is convenient to have marked on each post the height of the form for successive lifts. Each height will be 4 in. less than the depth of the form, so that the form will always lap 4 in. on the concrete below. The first marks should be established accurately with a level Two pieces instead of one are used for the

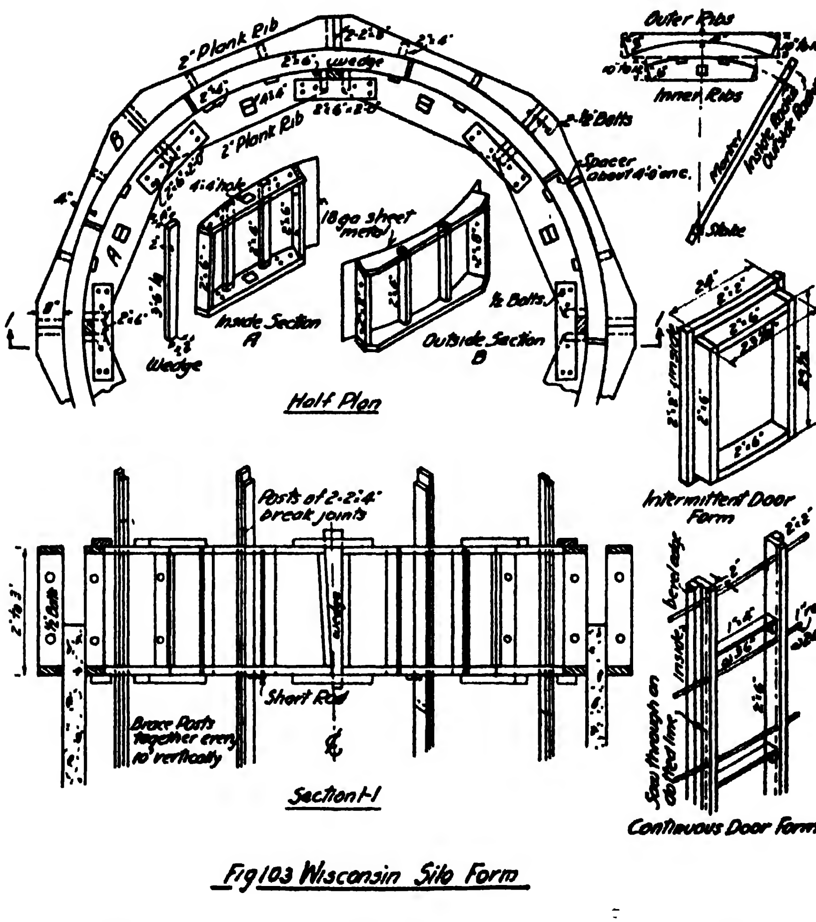


Fig 103 Wisconsin Silo Form

posts, so that they can break joint when splicing on, the first posts consisting of a long and short piece. The inside posts can be braced together to carry scaffold planks.

When the first pour of concrete has set (this can be the following day), the outside form is unbolted, raised, blocked in place, and the bolts tightened up. The reinforcing steel for the next lift is then placed and the inside form unbolted at the quarter points, the wedges removed, the sections raised, bolted together again, and wedged into place. The operation is repeated until the top is reached.

Short rods placed in holes in the posts under the lower ribs will hold the form in place before bolting and wedging.

The outside form is blocked from outside scaffolding. The forms can be raised in several ways, the commonest being by block and tackle suspended from cross-arms nailed to the posts. Another method is to jack up from the scaffolding, but this is much slower. One lift a day is poured, the form being raised on the following day, so that at least 12 hours will elapse before stripping. After each lift the posts should be plumbed and the forms cleaned and preferably oiled.

Sometimes the form is built without any outside ribs or uprights, the galvanised iron being connected at the quarter points by bolts through lugs riveted near the edges of the sheets. This, however, does not give such a stiff form and is liable to get out of shape easily, especially if the concrete is not poured evenly around the wall.

In building silos it is necessary to provide door forms, either for intermittent or continuous doors. The construction of these is shown in *Fig.* 103. They are nailed to the main form, and should be bevelled for easy stripping. The 1 in. round rods shown in the form for a continuous door are for the purpose of reinforcing steel, ladder rungs, and for attaching the doors. They are placed through holes 2 in. from the face of the form, and the form is sawn through on the line of the holes so that it can be stripped in two parts, or a 2 in. by 2 in. can be lightly nailed to a 2 in. by 4 in. While sheet metal is more easily fitted to the curvature in small structures and gives a smoother finish, it should be compared with the cost of wood sheathing when the diameter is large.

In building structures of greater diameters than silos, instead of using the posts to keep the walls plumb, two lifts of forms are built, each about 3 ft. high (*Fig.* 104). The forms are bolted together through the ribs, and made more substantial than silo forms. When the two sections are filled and the concrete in the lower section has had at least 12 hours to set, the lower form is unbolted a section at a time and reset on top of the upper form, removing and resetting the outer form first. One form is then always in place against the concrete supporting the upper form, being held there by friction on the concrete only.

Continuous-Sliding Forms.

On a large job considerable time can be saved by designing the forms so that they can move continuously during concreting. There will be no time lost in stripping and resetting the forms. An ordinary moving form means using a small gang of handy men. If labourers have to be used for concreting, carpenters for the forms, and ironworkers for the reinforcing steel, only one gang can work at a time, as no one operation is continuous. This would be a decided disadvantage on a large job if there were no other work to do.

With continuous-sliding forms (*Fig.* 105), after the forms are once made and set carpenters are not needed except one to watch the forms and to make repairs, and concreting and placing steel go on continuously.

There will be a greater initial outlay as the forms will cost more, and in addition there will be the rods or pipes and jacks to buy, though the latter may sometimes be hired, and the cost of lighting the work at night.

Continuous pouring, day and night, is an advantage structurally, especially when watertightness is required as with oil and water tanks,

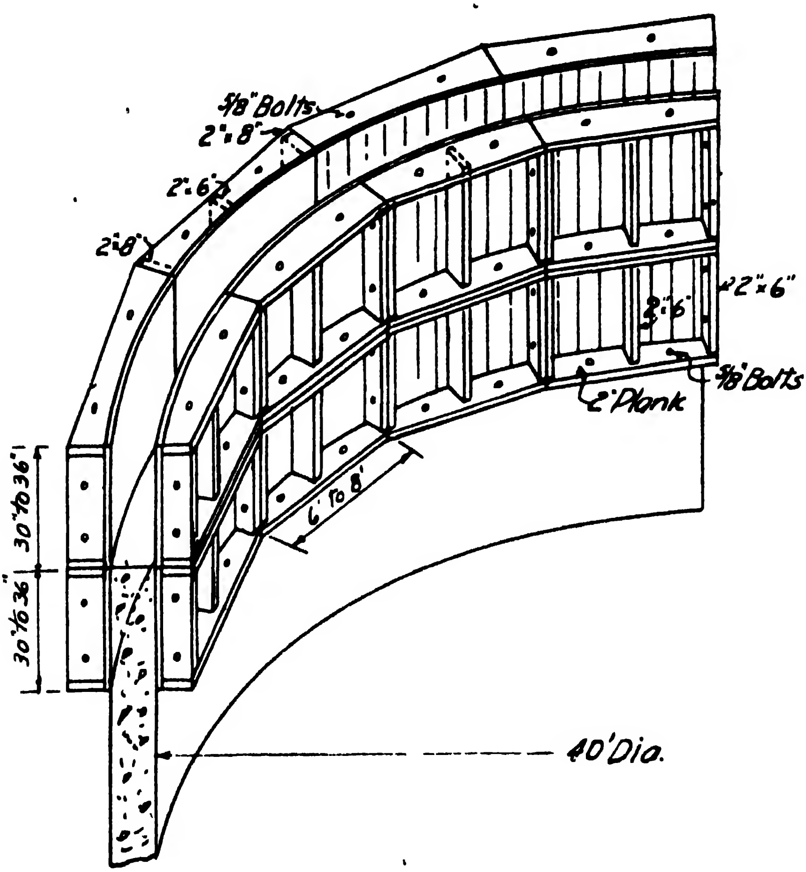


Fig. 104. Moving Forms for Large Diameters

but from a contractor's point of view the organisation of the work may be difficult as it will be necessary to have two or three shifts working. This, however, is mainly a question of the amount of labour available.

The various parts of a sliding form are the ribs and sheathing, making up the form proper, the yokes from which the forms are suspended, and the jack-rods or pipes and jacks by which they are lifted. Although the details may be built according to individual preference, the main principles are the same in all designs.

The height of the form may be 4 ft. to 7 ft. depending on the volume of

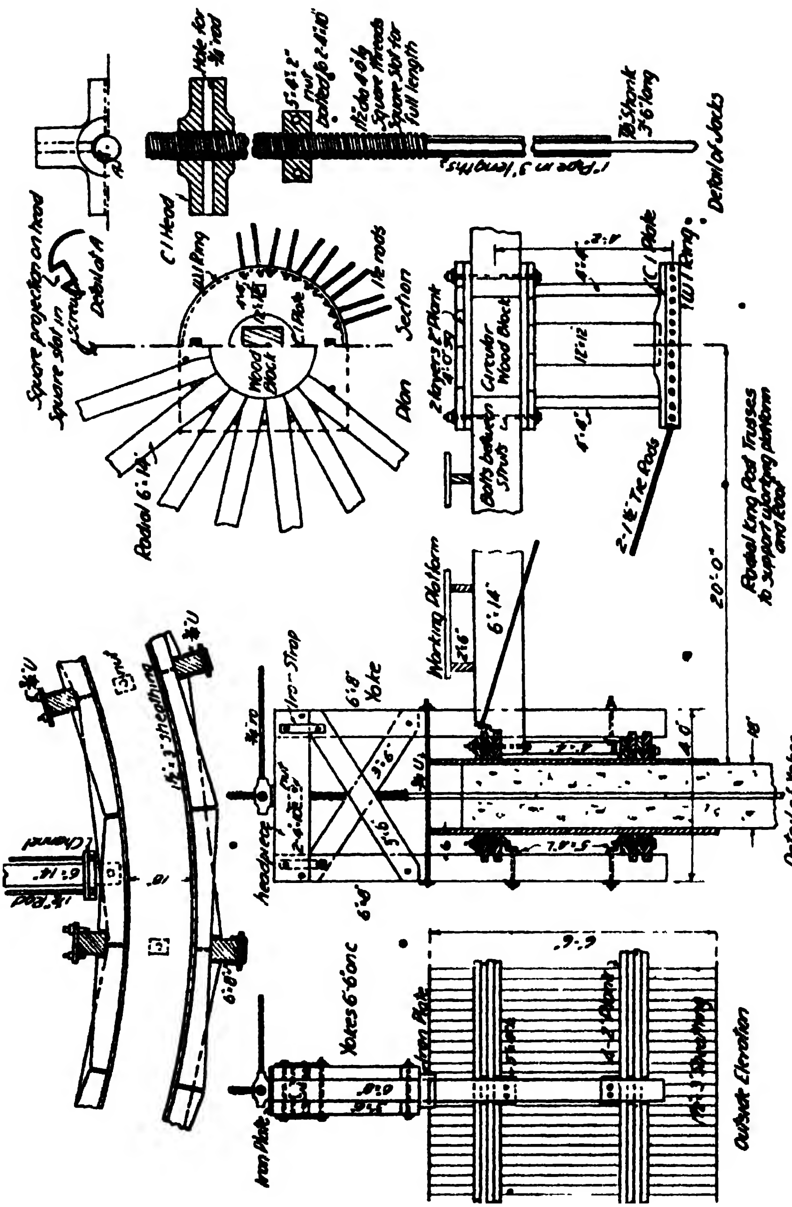


Fig. 185 Sliding Forms for Large Water Tank

concrete and speed required, and should be such that the concrete has at least 24 hours to set before being exposed.

The ribs should be 3 ft. to 3 ft. 6 in. on centre, and should consist of two or four thicknesses of 2 in. plank nailed together so that they break joint at the centre of each piece.

The sheathing, 1 in. to 1½ in. thick, can extend 12 in. to 18 in. beyond the centre of the ribs.

Unlike moving forms, the sheathing and ribs are built up into one continuous form, instead of in sections bolted together, except that the form may be sawn through at four points and bolted together to facilitate stripping and lowering the forms when the top is reached. It is best to assemble the ribs, one on top of the other, in their correct location, adjusting them to the circle before nailing on the sheathing.

Raising the top rib first to its right height and nailing on a few vertical boards and braces to hold it in place, the lower rib can then be raised to its position and the sheathing nailed on. If the wall is not thick enough to enable nailing to be done from the inside, the form can be assembled in sections, leaving off a few boards at the joints to allow for adjustment. The assembled form should be plumbed and levelled around its circumference, and spacers placed between the forms to give the thickness of the wall.

The next operation is to attach the yokes. These are designed in various ways. As the forms are suspended from the yokes, strength, rigidity, and positive connections are the main requirements. They should be placed about 5 ft. to 6 ft. apart, occurring at alternate joints in the planks. Yokes consist of two uprights, a headpiece, adequate cross-bracing, and means of attaching securely to the ribs.

A good method of construction is shown in *Fig.* 105. This design was made and used for the construction of a standpipe with an inside diameter of 40 ft. and a height of 100 ft., with 18 in. walls. Two of the four planks of the ribs are notched out for the 6 in. by 8 in. uprights, which extend about 3 ft. 6 in. above the sheathing. The uprights are bolted to the ribs with 5 in. by 4 in. angles. A ¾ in. U bolt holds the uprights together just above the sheathing. All connections are made with ¾ in. bolts. The headpiece of two 4 in. by 10 in.'s is bolted through iron straps to the uprights and carries the heavy nut through which the jack works. The nut is 5 in. by 4 in. by 2 in. thick, with square threads and two bolts for attaching to the headpiece. If one solid piece instead of two is used for the headpiece the nut is bolted on the underside. The yoke is sometimes made out of angle-iron to give greater stiffness, though heavy timbers will answer the same purpose.

There are two common types of screw jacks. The jack shown in *Fig.* 105 consists of an upper threaded portion 1½ in. diameter and 4 ft. long with three square threads to the inch, and a lower shank ⅞ in. diameter and 3 ft. 6 in. long, all in one piece. For the full length of the threaded portion a square slot is cut in the circumference, as at *A*. The turning head is a casting with four arms bored to take a ¾ in. bar and with

a 1½ in. diameter hole in the centre so that it will slip over the jack. There is a square projection in the side of the hole that will fit the slot in the jack. The head is therefore free to slide up and down, but cannot turn sideways without turning the jack. This arm always rests on the iron plate across the headpiece. The shank of the jack is inserted in a plain piece of 1 in. pipe 3 ft. long, and the pressure of the jack is transmitted by the collar at the junction of the threaded portion and the shank. As the

FIG. 106.—CONTINUOUSLY SLIDING FORMS, USED IN CONSTRUCTION OF STANDPIPE. FORMS HAVE JUST COMMENCED TO MOVE.

headpiece is turned with a bar, the jack working within the nut attached to the yoke and pressing on the pipe will raise the form.

The other type of screw jack commonly used consists of a similarly threaded hollow shaft which slips over a rod, with a clutch at the bottom of the threads to grip the rod. The turning head may be similar or may be attached to the top of the screw, although it is an advantage to be always able to apply the pressure at the top of the yoke where the jack has the greatest stiffness.

The procedure of jacking with the hollow jack is the same, but in this case successive lengths of rod are used instead of pipe, connected together with sleeves. One-inch diameter rods should be used.

When ready to pour, a length of pipe reaching from the foundation to the underside of the headpiece is placed at each yoke, the shank of the jack inserted in the pipe, and the pipes are plumbed.

When sufficient concrete has set in the forms to hold the pipes firmly and to guide the form, the form can be raised by giving each jack a quarter turn. After once starting to raise the form it should be carried on continuously, the rate of raising being such that the exposed concrete has had about 24 hours to set. From the start the speed can be increased a little each day until the maximum is reached. When the turning head has

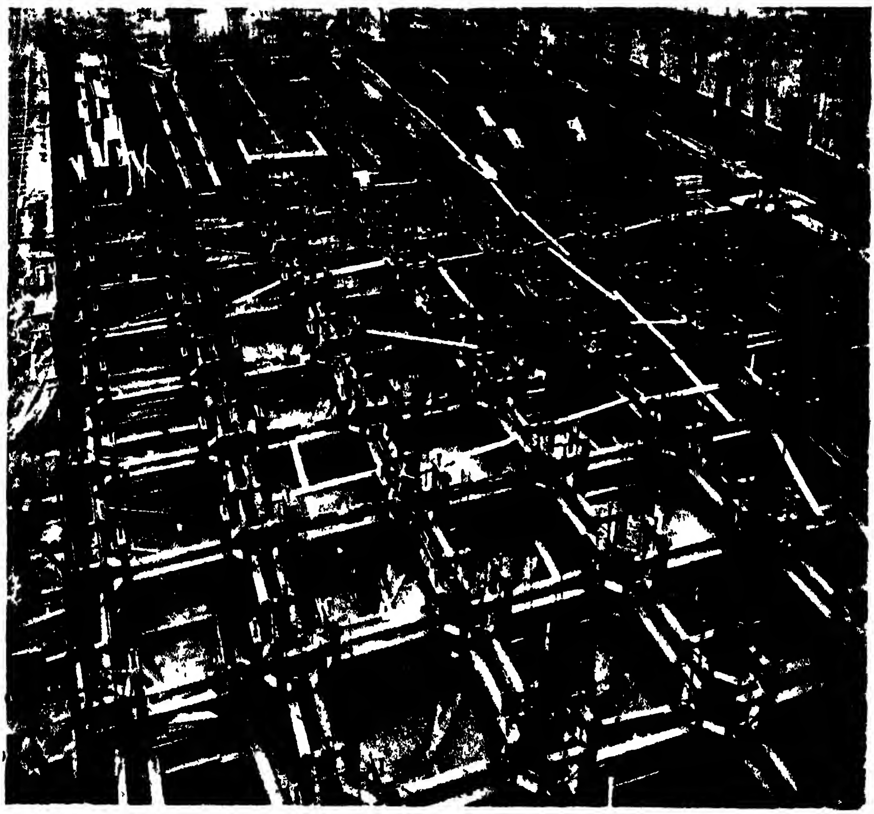


Fig. 107. Moving Forms: the King George Dock, Hull.

[Showing jacks assembled at each corner of each bin, with two rows of walings around each wall. In the background the vertical shuttering and the beam boxes to form the concrete beams of the top floor are seen in place. The decking is not yet laid. The hopper bottoms are shown. All the plant in this view ascends as the concreting proceeds.]

reached the top of the jack the jack is screwed up and a 3 ft. length of pipe placed on top and in line with the pipe in the concrete. The shank is placed on the pipe, and being 6 in. longer will extend for that distance into the pipe below, thus keeping the successive lengths of pipe in line. The operation is continued until the top of the structure is reached. It is essential that the jacks be operated at a steady and uniform rate and that each jack is turned the same amount so that the form will remain plumb. *Fig.* 106 shows the same standpipe just after the forms had commenced to move.

It is not necessary that all the jacks be operated simultaneously. Instead, one man can handle a group, going to each one in turn. Any uneven raising or stoppages will cause the concrete to stick and be dragged up to some extent with the forms, causing pockets and voids. On a large job where there are many jacks, each group should be operated at a given signal from the man in charge.

When using jacks on a battery of rectangular bins, there are usually four jacks at the corners, one in each bin, and there should be one in the centre of each side if the span is long (*Fig.* 107), which shows the sliding forms and jacks used in the construction of the King George Dock at Hull. As the forms are raised they should be checked for being level and plumb. The essential features of the jack are that it should permit of positive and uniform control and that there will be no slipping

Scaffolding.

With ordinary moving forms, if only one lift is used the scaffolding is carried up inside and out from the ground With two lifts, where one is raised on top of the other, it may be suspended from the forms. With

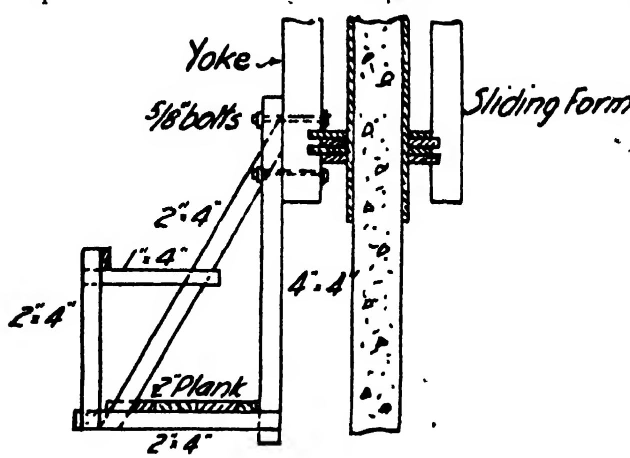


Fig 108a Hanging Scaffold.

continuous sliding forms the scaffolding is always suspended from the yokes (*Fig.* 108).

The concrete is finished as the forms are raised to avoid swinging scaffolds from the roof, so that it can be finished as soon as possible after stripping.

Concreting on small jobs is done by a small derrick erected on the forms ; on large jobs generally by chutes. Reinforcing steel is placed just ahead of the concrete.

On very wide structures a working platform can be built out from the inner form. In *Figs.* 105 and 106 the working platform is built over trusses which are attached to and move up with the forms. In *Figs.*

107 and 113 the working platform is the roof form which also moves up with the wall forms.

Roofs.

Roofs over these structures are of three kinds: flat, conical, or dome shape. With very high structures, especially when the diameter is large, it is often a problem to know how best to support the roof forms. There are three ways of doing it: by scaffolding from the bottom, which can

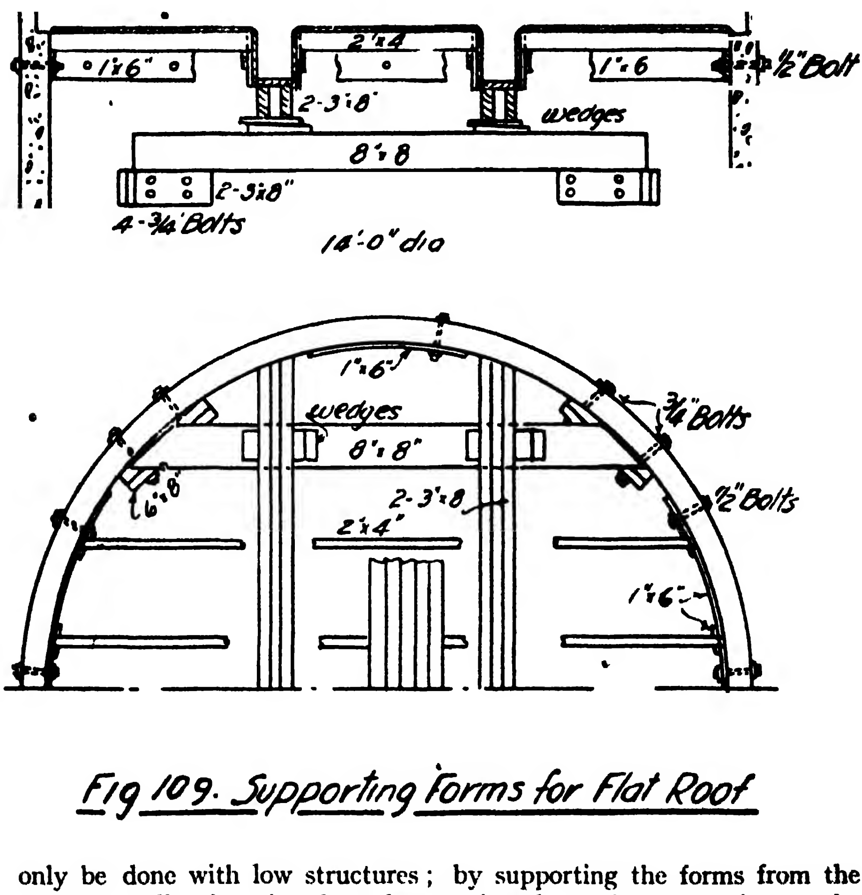


Fig 109. Supporting Forms for Flat Roof

only be done with low structures; by supporting the forms from the concrete walls after they have been stripped; or by supporting on the wall forms when they have reached the top.

Forms for diameters up to about 20 ft. when the roof is flat, can be conveniently supported on the concrete. The method consists of carrying the formwork on timbers, supported by wooden blocks bolted to the wall. Having decided on the method before the last lift has been poured, iron sleeves or round tapering wooden pegs are set in the form to take bolts after the form is stripped, spacing them as required. Short blocks, which may be a single thickness or built up, are attached to the bolts.

The bolts should be proportioned for shear and bearing stresses. These blocks carry the timber framing for the forms. Several different arrangements can be used; one for a beam and slab roof is shown in *Fig.* 109. The arrangement should be such as to cut down the spans as much as possible and to distribute the load around the wall.

For longer spans the supporting members may be trussed. When pouring the slab some eye-bolts should be set, projecting below the slab and anchored into the concrete, from which to hang scaffolding for stripping the forms, not forgetting also to leave an opening in the floor.

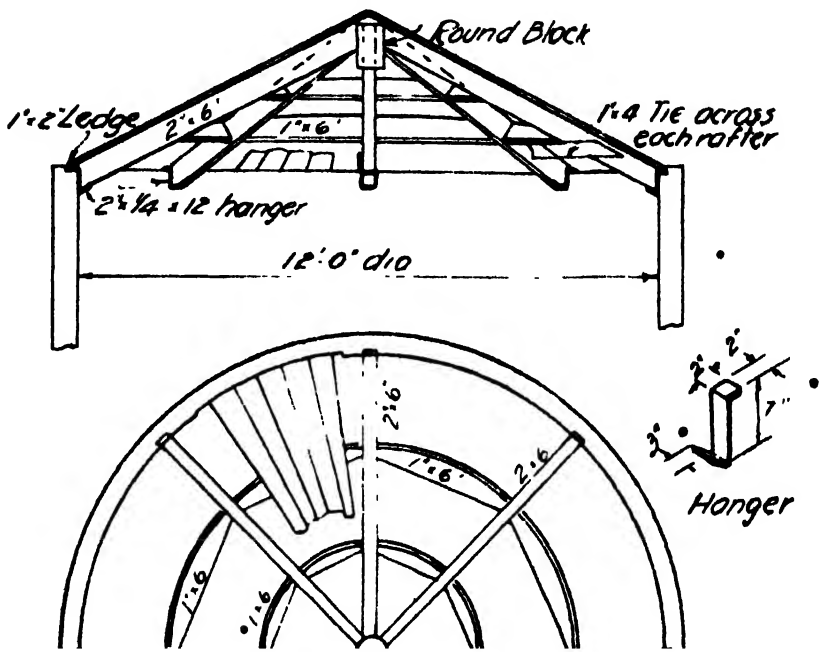


Fig 110. Supporting Conical Roof for Silos

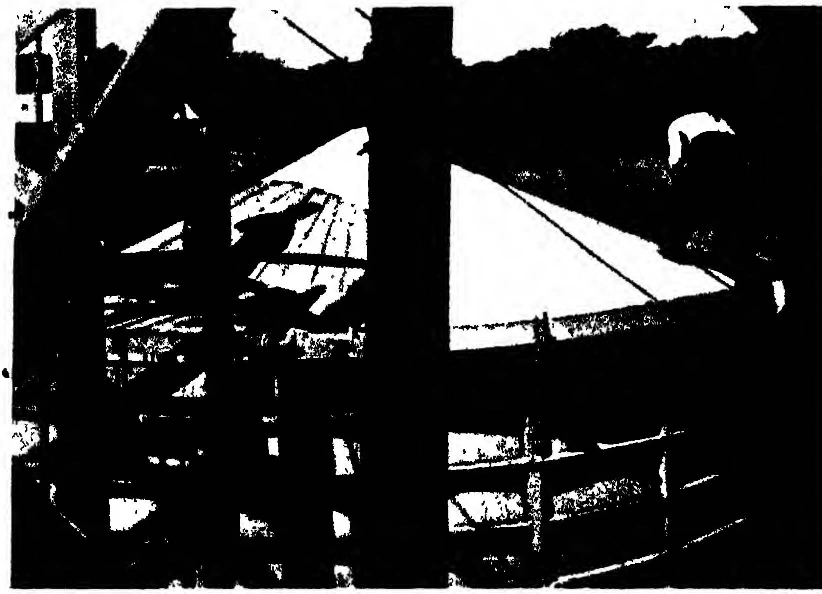
With flat roofs it will usually pay to use structural steel beams, as these can be set in the walls and the forms carried from them.

Conical roofs are usually used to cover silos. The maximum diameter will be about 20 ft. and more often will be about 14 ft. A framework is built up of 2 in. by 6 in. rafters meeting at the centre and braced with curved 1 in. by 8 in.'s about 30 in. apart. The rafters should divide the circumference into equal parts so that they are 5 ft. to 6 ft. apart. The lower ends of the rafters can be supported on ½ in. by 2 in. hangers about 12 in. long and bent as shown, or can be carried on the inner form, this being dropped sufficiently and held in place by the wedges and short rods

through the posts. The sheathing must be cut in triangular strips of length equal to the side of the cone. The lower end of the sheathing can be supported on the concrete by leaving a 1 in. by 2 in. ledge when pouring the top lift (*Figs.* 110 and 111).

Domes are used to cover structures of large diameter, such as standpipes, the diameter being as large as 40 ft. to 60 ft. The forms cannot be easily carried on the concrete, as the ribs have to be supported at several points.

In the standpipe shown in *Fig.* 105 the dome forms are supported by a



CONICAL ROOF FORMS FOR SILO.

series of radial king post trusses attached to the sliding form, one truss halfway between each yoke. The trusses consist of top compression members of 6 in. by 14 in. timbers, bolted to the upper rib of the sliding form and supported on the lower ribs by 4 in. by 4 in. posts, and also bolted to a built-up centre compression member common to all trusses, and two tie rods threaded each end (one each side the timber), passing through a channel butting against one end of the timber and passing through a wrought-iron ring under the casting of the centre member. This ring is loose to adjust itself to the tension in the rods.

The trusses were carried up with the forms and a working platform built over them, the sheathing afterwards being used to cover the dome. The dome form consisted of a series of vertical curved ribs of double 2 in. plank, meeting at the centre, with curved plank cross-braces about 3 ft. apart between them. The ribs were supported by 4 in. by 4 in. posts

from the trusses, the posts being cross-braced. The sheathing cannot be laid on regularly, owing to the curvature, but is used to the best advantage as shown in *Fig.* 112, which shows the dome form covering the standpipe. No outside forms were necessary as the concrete was poured fairly stiff.

An interesting example of roof forms for square bins, carried up with the forms, is shown in *Fig.* 113, which is a later view of the forms shown in *Fig.* 107.

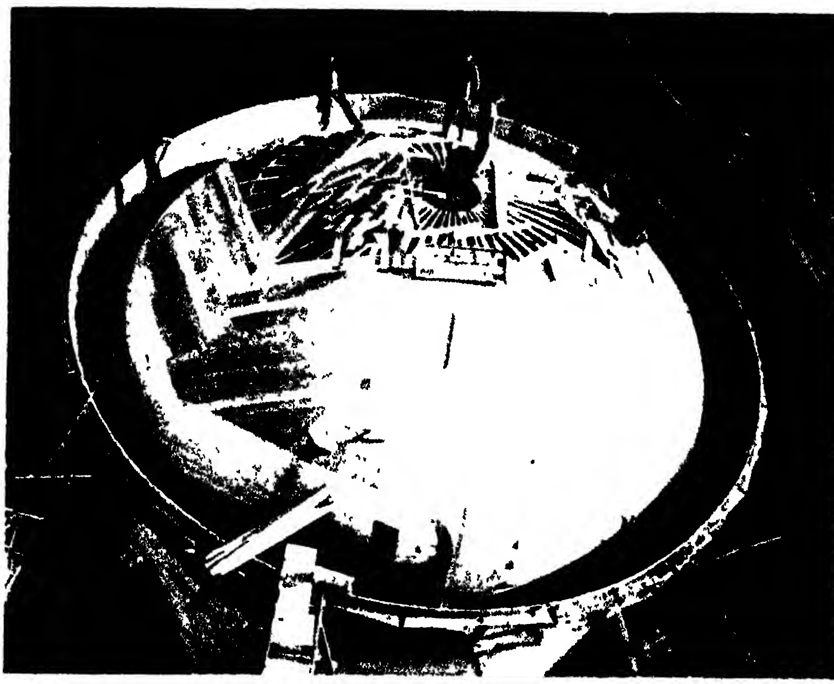


Fig. 112.—Dome Forms for Il[illegible]on, N.Y., Standpipe.

Estimating Cost.

Whatever system is used the cost of material can easily be estimated from a sketch of the proposed form. The cost of making up the form will depend somewhat on the diameter, as the forms must be made heavier and more carefully as the diameter increases.

Five carpenters per day of 8 hours should make up 100 cu. ft. of timber into circular forms.

Continuously sliding forms will cost about the same as moving forms for the ribs and sheathing only, but there will be additional labour cost for making and setting the yokes, setting jacks and jack pipes or rods.

With moving forms the same men will usually perform all operations and it is not necessary to separate the cost of moving the forms from the cost of concreting and setting steel, etc.

Since a moving form is poured its own height each day the total number of days required is easily estimated, and so many men will be required each day depending on the size of the job. The number of men

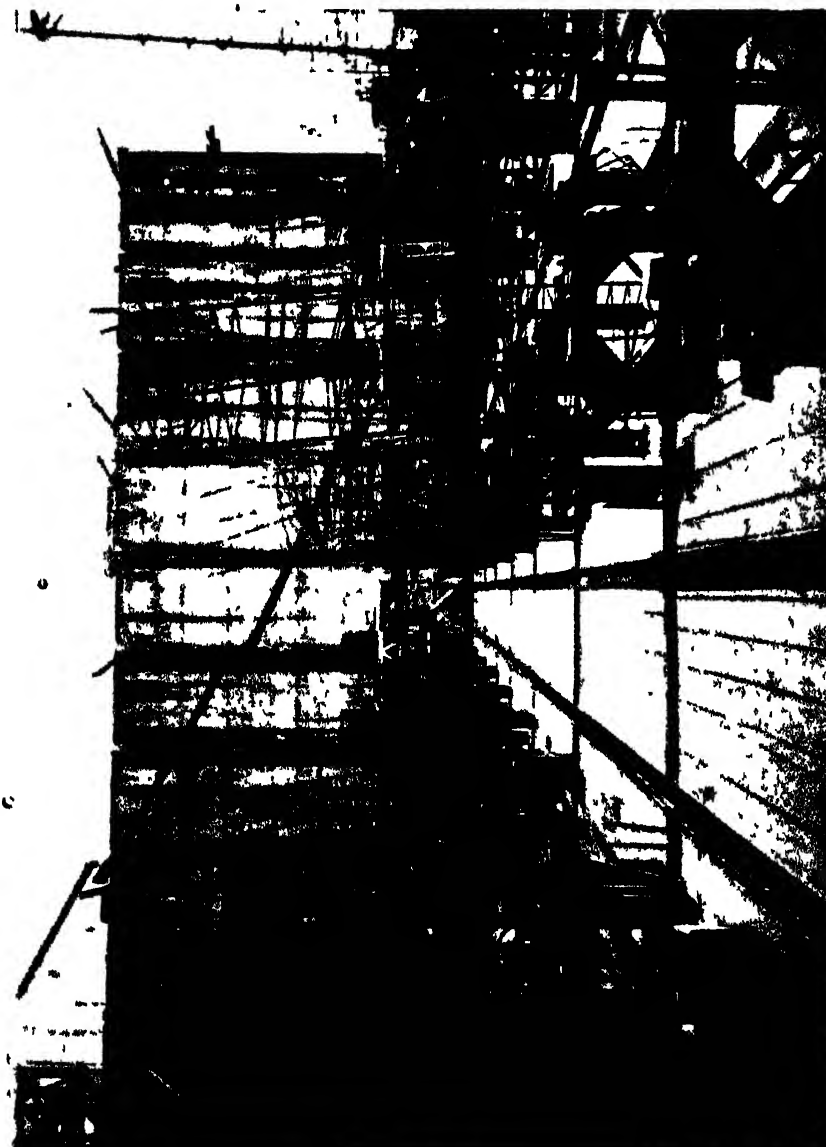


Fig. 113 Moving Forms at the King George Dock, Hull

[Showing the jacks and decking complete The eight concreting wagons the tran-porter wagon and the chute from the concrete tower are

will be governed by the labour required for concreting, four to five being sufficient for small silos.

Continuously sliding forms will require a certain number of men on the forms all the time to manipulate the jacks. One or two men will take care

of all the jacks on a single structure. On multiple-cell structures boys can often be used, each boy handling so many jacks under a foreman. Since a maximum of 6 ft. will be poured in 24 hours, the rate of raising the forms will hardly ever exceed 3 in. per hour. If one complete turn of the jacks raises the form $\frac{1}{3}$ in., nine turns per hour will be necessary, and this determines the number of men required.

In the construction of the bins shown in *Fig.* 107, each boy gave a complete turn to 19 jacks in about 10 minutes, the forms being raised about 2 in. per hour. The forms shown in *Fig.* 106 were raised about $2\frac{1}{2}$ in. per hour.

CHAPTER XV.

FORMS FOR DAMS, PIERS AND HEAVY WALLS.

DAMS are of two main types the gravity dam, which is the commonest; and the hollow type, or reinforced concrete dam, which will often prove more economical For the purpose of describing the form construction, gravity dams can be divided into low and high dams. High dams and bridge piers and heavy dock walls can be built with similar methods, providing horizontal joints are not objectionable.

Low Gravity Dams.

Gravity dams only 10 ft. or 12 ft high are formed to the top, because the yardage of concrete per foot of dam is small and 50 to 100 lineal feet of dam can be poured complete at a time. The forms will generally only be used once, or at most two or three times, so they need only be made strong enough to take the pressure, depending on the height of the dam.

The downstream face is generally curved top and bottom, if it is straight it will be built similarly to the upstream face. *Fig.* 114 shows typical forms for a small dam and the method of bracing them. The two planks forming the curved ribs should be nailed together and the lower planks sheathed before placing This can be done in sections 10 ft. or 12 ft. long, as it would be difficult to place the sheathing after the ribs are set. Instead of sheathing in sections the ribs can be placed in position, supported 3 ft. or 4 ft. above the footing, temporarily braced, and the sheathing nailed on, the whole form then being lowered to the footing. The remainder of the sheathing is nailed on in place The sheathing for the curve at the top should generally be left off until required, in order to give a larger opening through which to place the concrete.

As there will be considerable upward pressure on the lower curved form, it is advisable to weight it down with rocks or bags of sand to relieve the tension in the wires. The lower parts of the forms are held by wires or bolts, which are anchored around ¾ in. rods embedded in the concrete near the bottom of the footing, which is poured ahead.

The upper wires or bolts go across from wale to wale. Two strands of No. 9 wire are satisfactory for heights to 10 ft., but care must be taken that they are not broken when placing "one man" stones.

A 2 in. by 6 in. stiffener should be placed as shown on alternate studs, with a short piece nailed across to the curved rib in order to hold the form in shape. The form can be stripped in two to three days.

The cost will be about the same as that given for straight and curved

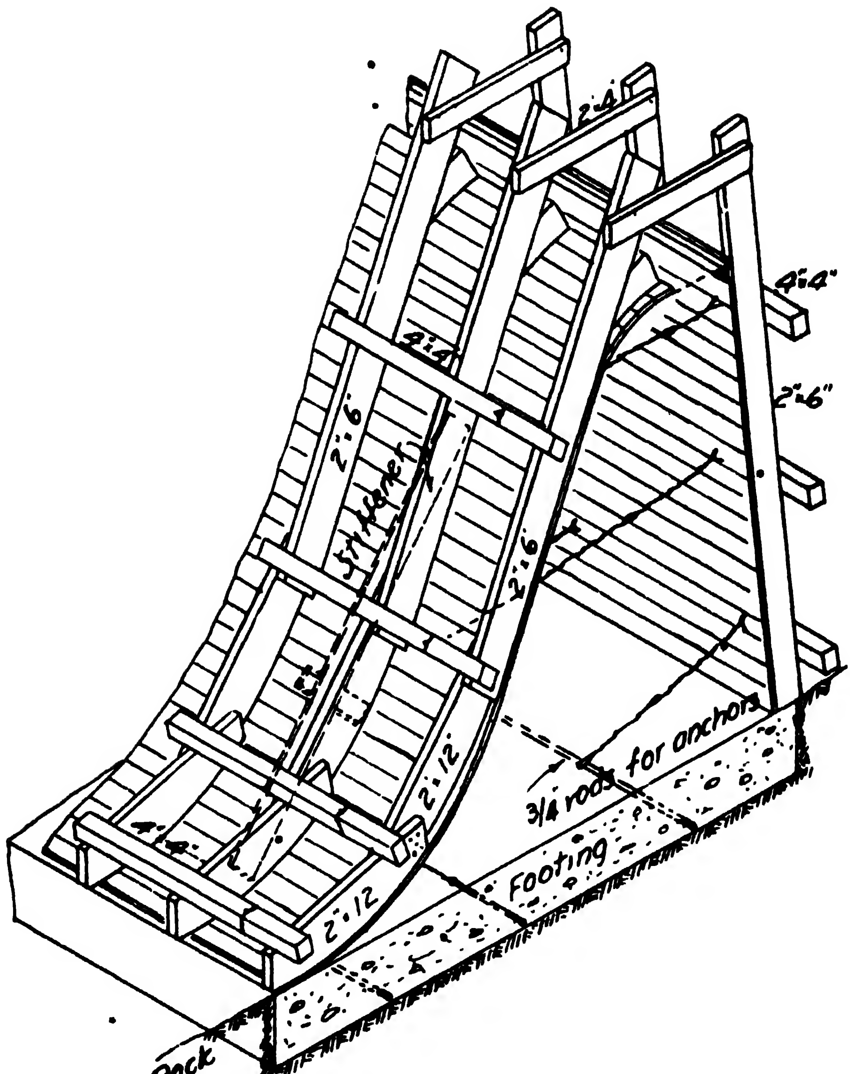


Fig 114. Forms for Small Dams

walls in Chapter VIII when there are no special difficulties; but if it is built within a cofferdam the cost will be higher, as it will be more difficult to carry timber to the site and the erection may have to be done in a certain amount of water.

High Gravity Dams, Piers and Heavy Walls.

Dams are always, and piers and heavy walls are usually, built with panel forms, using two tiers, lifting one up above the other until the top is reached. The construction joints are vertical, the structures being poured in vertical sections about 30 ft. long.

The formwork is simple, the main problems being the economical lay-out of the panels and the method of holding them in place. As the yardage of concrete is very large compared with the area of forms required, it is not necessary to make the panels very high, 8 ft. to 10 ft. being suitable heights.

In the construction of isolated piers, sufficient panels are made up to go completely round the pier, two tiers high.

For economy in dock wall and dam construction, two, or better still three, adjacent sections should be constructed together. Specifications will sometimes call for the structure to be built in alternate sections, in which case two or three sections should be built together. When the sections are adjacent, each should be concreted a panel height ahead of the adjoining section By this means there are always two or three sections in which concrete can be poured, so the height poured in any one section will usually not be over 4 ft. to 6 ft., which will cut down the pressure on the forms. Near the top, where the width is small, a section may be poured to the full panel height, so the panels must be designed for that condition The panels, however, are always made heavier than theoretically required in order to withstand the wear and tear of repeated usage

An elevation of the structure should first be drawn, dividing it into the number of vertical sections to be poured, each about 30 ft. long. The height of the panels should then be decided upon ; this will depend on the height of the structure and the economical length of studding.

As studs can only be bought in multiples of 2 ft. long, as near a standard length as possible should be used, otherwise there may be as much as 20 per cent. waste in cutting. It will not matter if the top panel sticks up a little beyond the top of the concrete ; this will be cheaper than cutting the studs to make the top panel exactly flush with the top of the concrete.

The panels for the sloping side of a dam must be higher in order to bring the front and back forms to the same height, and will generally be about 25 per cent. higher than the panels for the vertical face.

The heights of the panels or successive pours should then be marked on the sketch of the elevation. Numbering the panels in the order in which it is desired to pour the sections, the number of panels that it will be necessary to make up can be determined, remembering that the lower tier of panels cannot be stripped until the upper tier is completed.

The speed in this class of construction depends largely on the speed of the carpenter work and the lay-out of the work, so that there will always be some forms ready for concreting.

The length of the panels should be such that two or three together will be a few inches wider than the width of the section, so that they can overlap the poured section when necessary and so that the end bulkhead can be placed inside the form.

A suitable size panel for handling will contain 80 to 100 sq. ft., weighing 600 to 800 lbs.

Bulkhead forms will be required at the ends of sections, the number of panels required depending on the order of pouring the sections. This

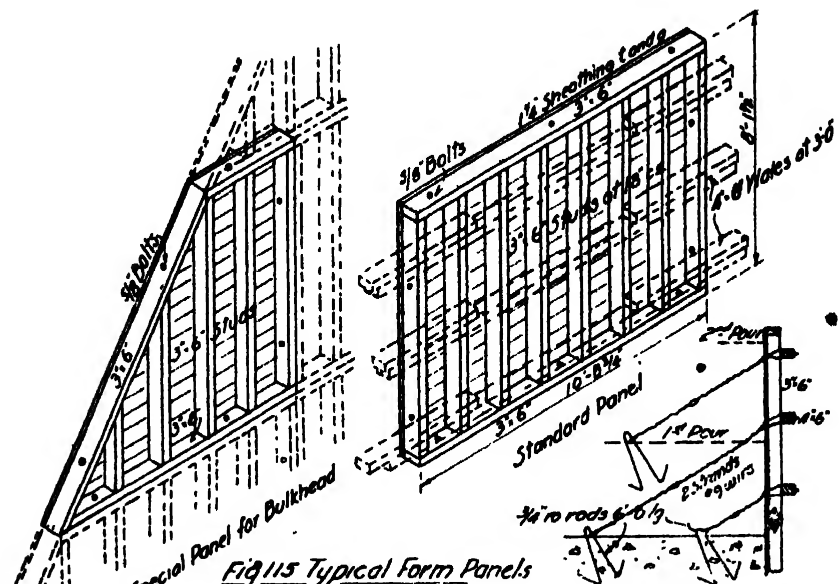


Fig 115 Typical Form Panels for Dams

Method of Holding Forms

form will be built to fit the cross section, fitting inside the side forms and bolted to them. Standard size panels, as used for the sides, are used together with special panels made to fit the slope.

Fig. 115 shows typical panels the author designed for a dam 400 ft. long by 48 ft. high, the width at the bottom being 34 ft. and at the top 8 ft. Forty-five panels, including special panels for the sloping bulkhead, were made up, using them 14 times over, this number allowing work to be carried on in three 30-ft. sections

Figs. 116 and 117 show the forms in use. *Fig.* 117 shows the valve house built with the standard panels, without cutting them down to the exact size required. Note also the scaffold brackets made to hook over the wales. The sheathing was 1¼ in. spruce tongued and grooved and dressed both sides; the studs and caps were 3 in. by 6 in. and the wales

FIG. 116.—FIRST SECTION OF DAM SHOWING FACE AND BULKHEAD FORMS.

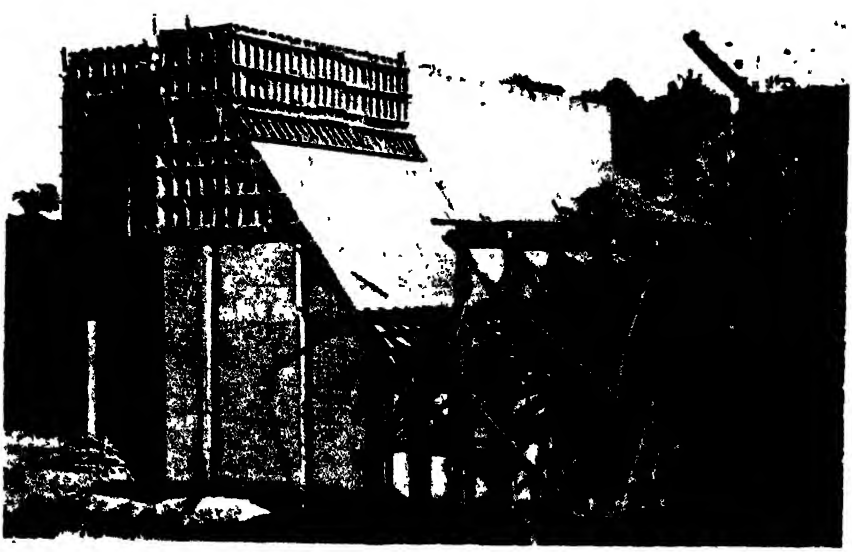
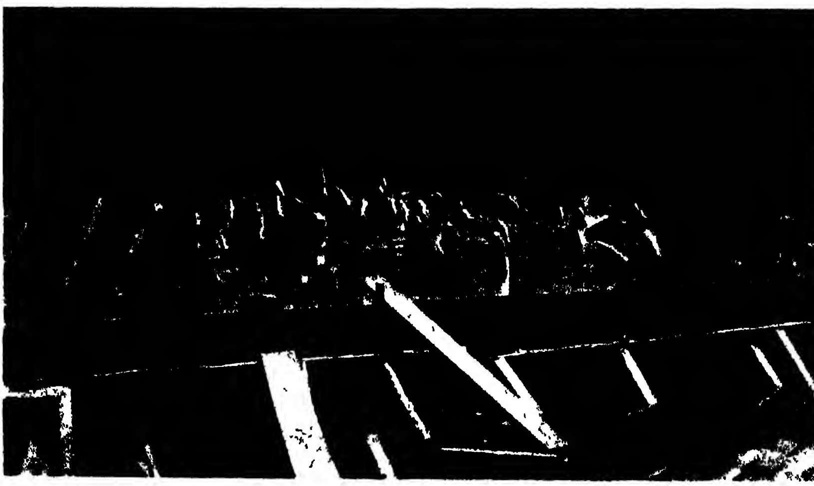


FIG. 117.—THIRD SECTION OF DAM, SHOWING GATE-HOUSE AND BACK FORMS.

4 in. by 6 in. yellow pine dressed four sides. These sizes are heavier than required for the maximum pressure, but were made to withstand repeated usage. The sheathing finishes flush with the outside of the studs and top and bottom caps. Three ⅝ in. bolts top and bottom, and two on each side, were used to fasten the panels together. There is little pressure on these bolts, as the pressure is taken by the wales. The special end bulkhead panels were built as shown. The wales were not permanently attached to the studs, but were loose and overlapped. If they are permanently attached, as they should overlap there would be more trouble in stripping the panels.

The method of holding the forms was the method usually adopted, of embedding "hairpin" anchors, to which the wales are wired, in the



concrete at each pour. These anchors are ¾ in. rods about 6 ft. long, bent as shown and projecting about 4 in. from the concrete. The angle the wires make with the horizontal should not be greater than 45 degrees. Two No. 9 wires for each tie are sufficient for a pour of 5 ft., and three wires for a pour of 10 ft. Bolts hooked around the anchors can be used if desired, but they are more apt to slip. *Fig.* 118 shows the hairpin anchors. Near the top, where the width is small, the form is wired across from wale to wale. The sloping side is strutted temporarily from the inside until there is sufficient concrete to support it.

As soon as a section is poured to the top of the panels, the lower panels are stripped and moved ahead or to an adjoining section. The forms in this case were handled by a travelling derrick, running on a trestle parallel to the dam.

Curved spillway forms were made up and bolted together in the same way, the studs being cut to the required curvature out of 3 in. by 4 in.'s;

which were nailed to 3 in. by 6 in. straight studs to form 3 in. by 10 in. ribs.

The cost of these forms will be considerably higher than for ordinary low walls, where most of the work can be done from the ground, as labour is less efficient when working at great heights. To make up 100 sq. ft. of panel forms will require about 8 hours carpenters' time and 4 hours labourers' time. To set, wire up, and strip the same will require about 8 hours carpenters' time and 8 hours labourers' time.

The forms must be oiled after each use, and this time is included.

Reinforced Concrete Dams.

These generally consist of a series of buttresses supporting a sloping slab. The buttresses, if high, are built with panel forms as just described;

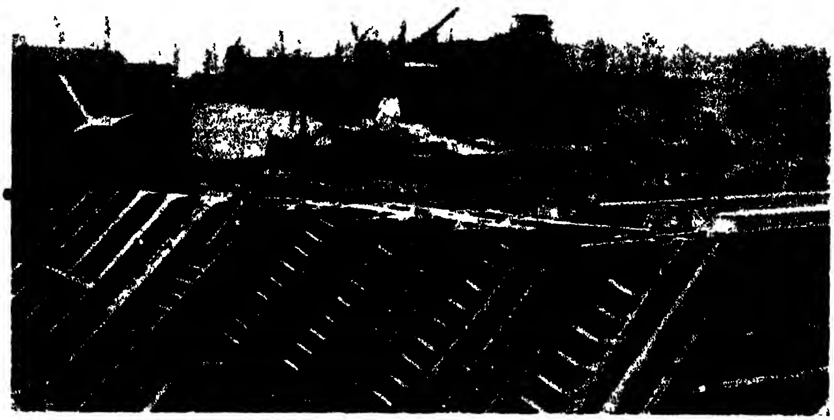


FIG. 119.—BUTTRESS AND BRACKET FORMS FOR A REINFORCED CONCRETE DAM

if low they are formed to the top complete. The top of the buttresses usually widens out to form a bracket on each side to support the slab, the buttress itself however being poured to the top of the slab.

The buttress and bracket forms are clearly shown in *Fig.* 119. The buttress is stripped before the slab forms are placed. The under-slab form is supported by joists heavy enough to span between buttresses, resting on ledgers bolted to the concrete. The bolts generally pass through pipe sleeves, which are set when the concrete is being poured. The top form for the slab rests on the upper part of the buttresses and is held down by bolts embedded in the concrete and passing through double wales. At the centre of the span the upper and lower forms are bolted together through double wales. Both forms are built in panel form. The construction is shown in *Fig.* 120, which is a later view of the same dam as shown in *Fig.* 119.

FIG 120.—SLAB FORMS FOR REINFORCED CONCRETE DAMS

The cost of making-up the panels will be the same as given for gravity dams. The cost of placing and stripping the buttress forms should be a little lower than for gravity dams, as the walls are narrow and require less labour to wire up. The cost of placing and stripping the upper and lower slab forms should be about half the cost of the buttress forms.

The amount of timber required in this class of work is best calculated from a sketch of the forms, allowing sufficient for waste in making up the panels.

CHAPTER XVI.

STEEL FORMS IN BUILDING AND WALL CONSTRUCTION.

THE use of steel forms has increased rapidly in the past few years. They can now be obtained for any type of structure and of any shape, and have replaced wooden forms almost entirely for certain structures. Their economy depends, of course, on the number of times they can be used. It would not be economical to buy steel forms for a job on which they could only be used two or three times, unless the contractor could move them from one job to another They should be regarded more as plant than material, and the same considerations should govern their purchase as with a piece of plant Interest on investment, depreciation, upkeep, storage, handling, freight charges, etc, should all be considered in addition to first cost As with any other plant, to be economical they must be in constant use

Where a job is sufficiently large steel forms will usually more than pay for themselves on the one job, even when perhaps owing to special construction they cannot be used in other work Steel forms can be hired instead of bought outright, though this practice is much commoner in the United States than in this country. The rental will usually amount to about 70 per cent of the purchase price, though this depends on the length of time used If hiring can show a saving over wood forms, even considering the salvage value of the timber, it may be the best policy, since the next job on which they could be used is always uncertain and storing is not profitable

Besides the first cost, there is the question of the saving of labour in erection and stripping. This is considerably less with steel forms than with wood forms, and it is this saving of labour cost that will decide whether steel forms are economical or not. Steel forms arrive on the job ready to place, without the preliminary work required with wood forms, so that the time and labour spent in making-up the forms is saved. The methods of connecting panels, bracing, and tying being already provided for, erection is almost mechanical and requires no special skill and may be done by common labour, so that the cost of erection and stripping will be much less than for wood forms. Better work can be done, as the forms are factory-made and fit exactly.

Labour cost can be more accurately estimated with steel than with wood forms, since it is more of a mechanical operation and the "personal equation" does not enter into account so much. The labour cost of

wood forms may vary considerably on two similar jobs using different carpenters and foremen.

Other advantages of steel forms are smoother surfaces, requiring very little re-touching, denser surfaces and hence less absorption of paint, and the smaller number of men required.

It is therefore not always a simple matter to compare the relative economy of steel and wood forms, as there are so many items that should be taken into account for a true comparison. They can be obtained for any type of structure, and through all degrees of complexity from the

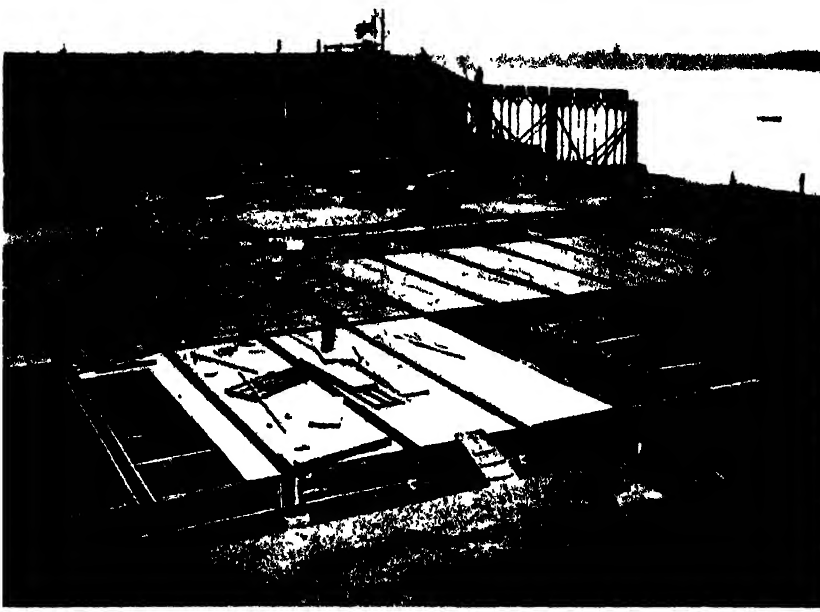


FIG. 121.—"DESLAURIERS" STEEL SLAB FO[illegible] IN BEAM AND GIRDER CONSTRUCTION.

simple light wall panels to the huge structural steel travelling forms used in dock-wall construction.

Although a contracting firm may design its own steel forms and have them made to order, it will usually prove more economical to buy them from the firms that specialise in their manufacture and who have had years of experience in their use. The forms manufactured by these concerns are of course patented. It would serve no particular purpose to describe the detail construction of the forms, since this can be obtained from the manufacturers, who will also advise the best type of form to use for a particular structure. Instead, by giving photographs of actual construction work in which they have been used, the multitudinous uses to which they may be put are shown.

It is convenient to divide steel forms into two groups, those for forming flat surfaces and those for curved surfaces. The former only will be considered in this chapter.

FIG 122.—"METAFORMS" WITH WOOD WALES

Buildings.—Steel forms for round columns and caps and for deck panels in flat slab construction have already been mentioned, as also have metal tile fillers, which are really steel forms. These are the commonest uses for steel forms in building construction.

Steel slab-panels for beam and girder construction have been used to some extent, in combination with wood beam boxes. Slab-panel forms manufactured by the Deslauriers Column Mould Co. are shown in *Fig.* 121 (note also the method of supporting the forms for a saw-tooth roof). Steel has not yet taken the place of timber for shores, joists, ledgers and beam sides. The only other extensive use of steel forms in buildings is for foundation and other concrete walls.

Light Wall Construction.—Light wall panels are 24 in. square, made of sheet metal with angle-iron stiffeners, fastened together by clamps, wedges, spikes, etc., according to the system used. They are made in standard sizes so that any length of wall may be formed, adjustable panels taking care of odd inches and fractions of an inch, and special corner pieces are provided. The usual method is to use two or three courses, one above the other, pouring 4 ft. or 6 ft. the first day and 2 ft. or 4 ft. on succeeding days, the forms being stripped in twenty-four hours. Using two courses, after the first two courses are filled the lower course is stripped and placed on top of the second course, forming the third course, and the second course forms the fourth course, and so on, one course always remaining in contact with the concrete to support the course above.

In the "Metaform" system the units are connected together with clamps. Using only two courses wales are not necessary, but they add to the rigidity of the form and facilitate the aligning. When more courses are used, adjustable steel aligners are clamped to the forms, a horizontal aligner to each plate and vertical aligners 8 ft. to 10 ft. apart. The aligners are made to telescope so that they can be adjusted to different lengths. Instead of steel aligners, ordinary 2's by 4's can be used with the same clamps. *Figs.* 122 and 123 show the use of "Metaforms" in light wall construction, with and without aligners.

In the "Blaw-Knox" system, Blaw light wall forms are connected together with wedges driven into corresponding slots in adjacent units. Angle-iron wales are dropped into slots on the panels, and vertical aligners are attached to these with wedges (*Fig.* 124). Steel bulkhead panels are provided.

Using the aligners with both systems enables a wall to be built to a height of 10 ft. at one pour, and also enables several panels to be moved as a unit, instead of one by one. In each system wire ties are used to prevent the forms from spreading, and metal spacers on the top course keep the forms the right distance apart.

The Blaw-Knox Co. manufacture angle-iron yokes which slip over and hold two courses of forms, if the use of wire ties is objectionable owing to the possible appearance of rust spots. The same company also manufacture a hand-operated machine for making the wire ties quickly and uniform.

It is essential that the footing on which the first course is set be level. No outside bracing will be necessary. The work of placing the forms can be done by common labour. The workmen should work in

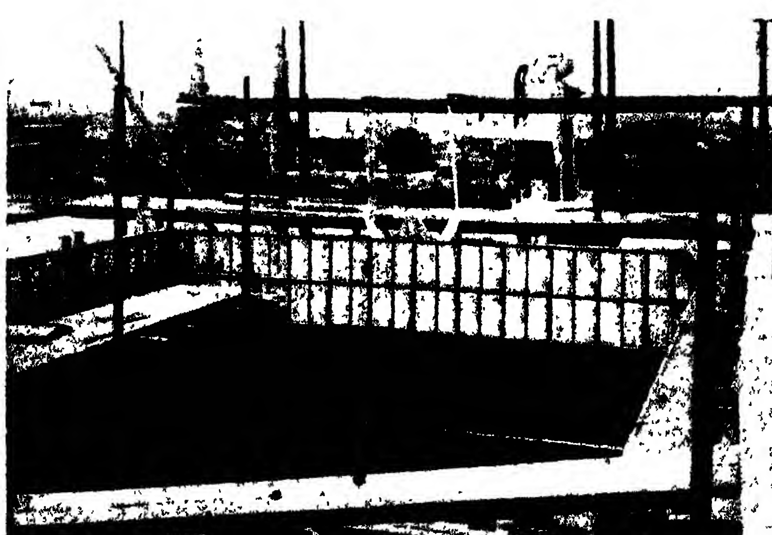


FIG. 123.—"METAFORMS" WITHOUT WALES.

FIG. 124.—"BLAW-KNOX" LIGHT WALL FORMS

pairs, one placing the outside panels and one the inside panels, the former working a little ahead of the latter.

The forms should be cleaned and oiled every time they are stripped, including the last stripping, even if they are not to be used again for some time. Remembering that they are part of a contractor's plant, they should have the same care.

Heavy Wall Construction.—On large contracts where there are heavy walls, as in docks and canal locks, and for heavy retaining walls, steel forms are particularly useful and economical. These contracts are

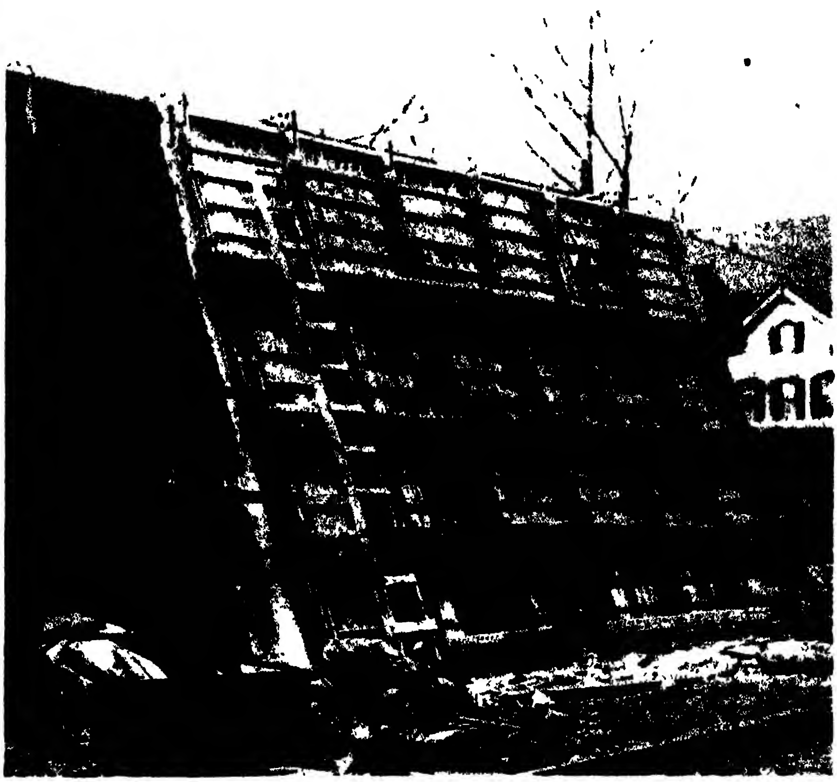


FIG 125—FORMS FOR RAILWAY RETAINING WALL, DESIGNED TO BE LIFTED IN ONE UNIT

usually so large that the first cost of the forms is less important than the saving in labour cost, and for that reason it is often economical to have steel forms specially designed and built for a particular purpose.

The method of raising one course above the other used in light walls would be entirely too slow and uneconomical on large work, so instead the walls are built in vertical sections about 30 ft. long, pouring the wall in one, two, or three operations This means that very heavy bracing is required, and that the bracing must generally be from the outside owing to the large width of the wall.

Figs. 125 to 132, showing the varied use of heavy steel wall forms,

are given by courtesy of the Blaw-Knox Co., of Pittsburgh, U.S.A. The forms may be designed as stationary or travelling. In the former case they are built up into units of such a size as can be conveniently handled by the plant available. A unit may consist of the forms for one side of the wall only, or for both sides.

In *Fig.* 125 is shown a form for a heavy railroad retaining wall, which is designed to be lifted in one piece and is also adjustable to various types of wall. *Fig.* 126 shows forms for both sides of a dock wall being lifted as one unit by a derrick

Fig. 127 shows forms for high thin walls with brackets at the top

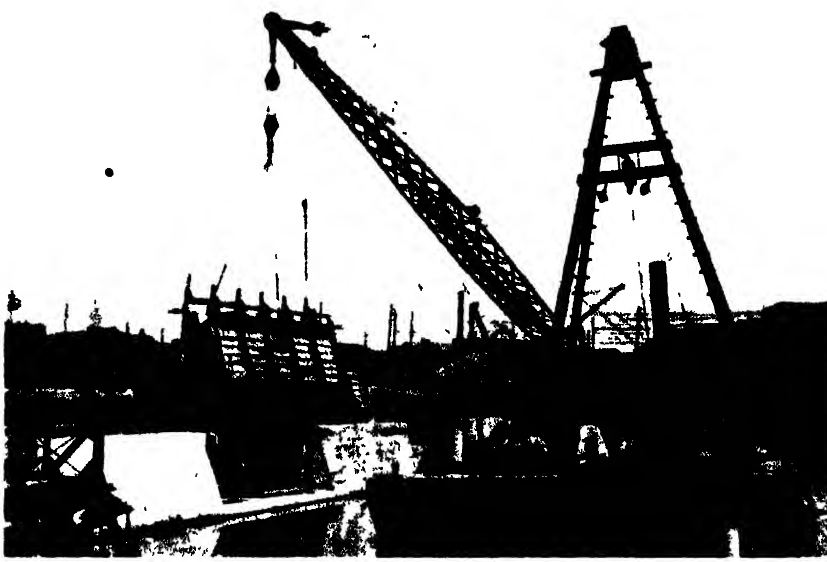


FIG. 126.—FORMS FOR A DOCK WALL. BOTH SIDES MOVED AS ONE UNIT.

being lifted in units by a cableway These forms were used in the construction of the Chicago Sewage Works. (Note the method of bracing the forms, and the large area of walls on which they were used.)

Instead of lifting the forms up bodily they may be designed to slide along horizontally. In this case they are attached to a structural steel frame called a "traveller." A unit of forms consisting of back and front forms may be attached permanently to the traveller, or the traveller may be detached and handle several units in turn. In the former case the traveller also braces the wall, while in the latter the wall forms are self-supporting. The travelling frame usually straddles the wall and moves on rails. The forms are collapsed by jacks, turnbuckles, or steamboat ratchets. The traveller with forms attached is moved by hand power for small installations and by winches for the larger jobs. For still heavier forms gasoline, steam, or electric hoists are used.

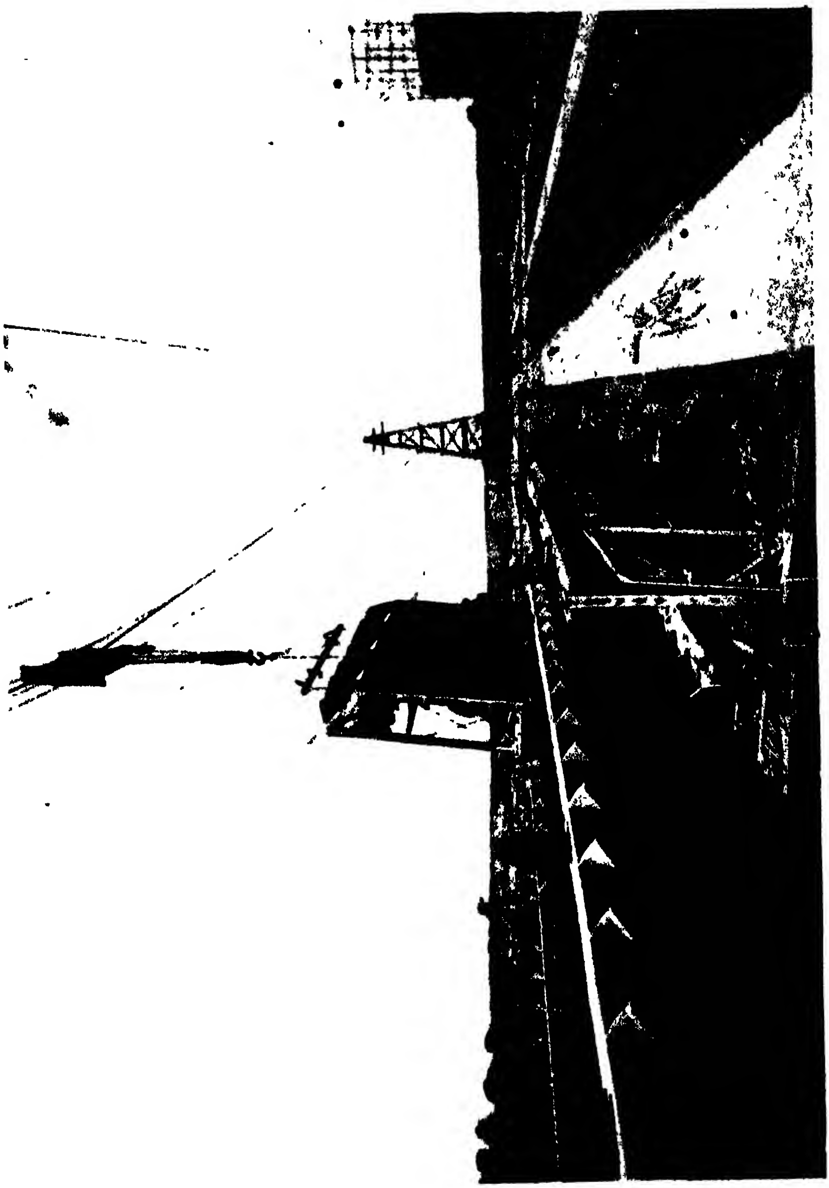


FIG. 127.—FORMS USED IN CONSTRUCTION OF SEWAGE TREATMENT WORKS: FORMS HANDLED BY CABLEWAY.

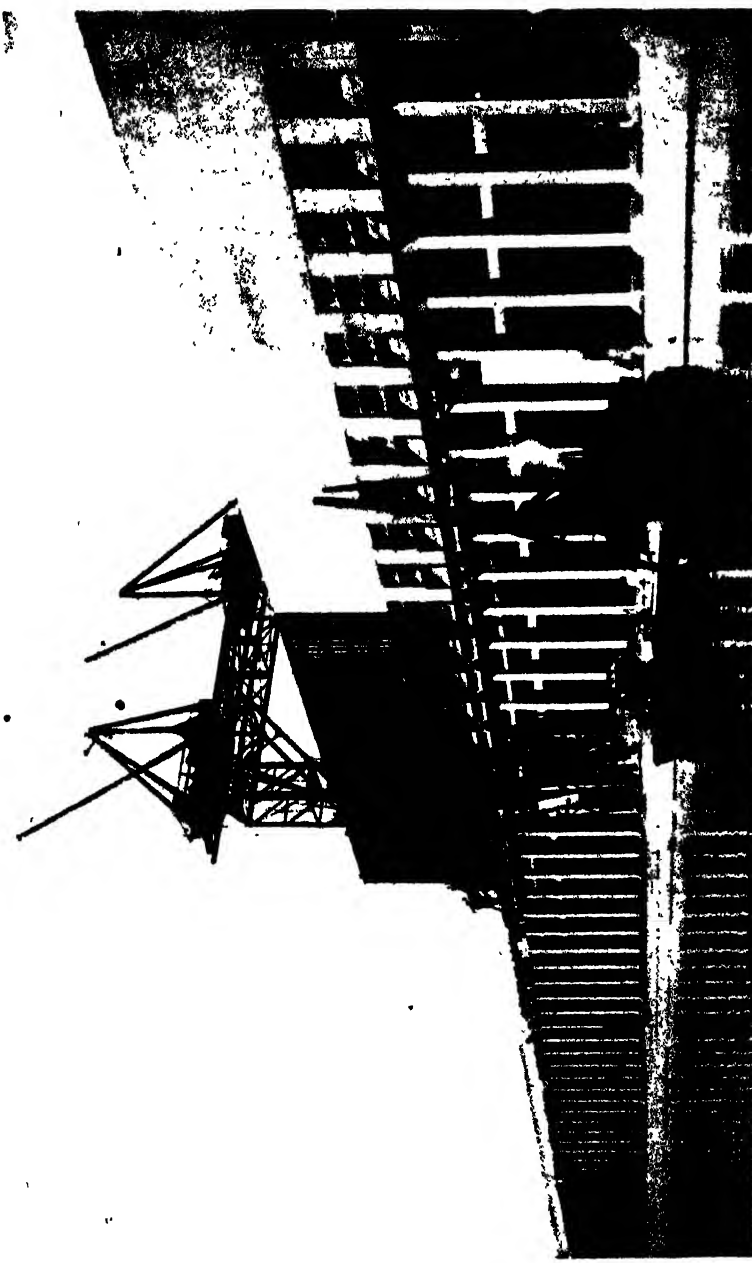


FIG 128.—HIGH TRAVELLER HANDLING INDEPENDENT PANEL FORMS IN CONSTRUCTION OF ONE DOCK

FIG. 129.—FIFTY FEET OF TRAVELLER HANDLING 150 FEET OF FORMS IN CONSTRUCTION OF SEA WALL.

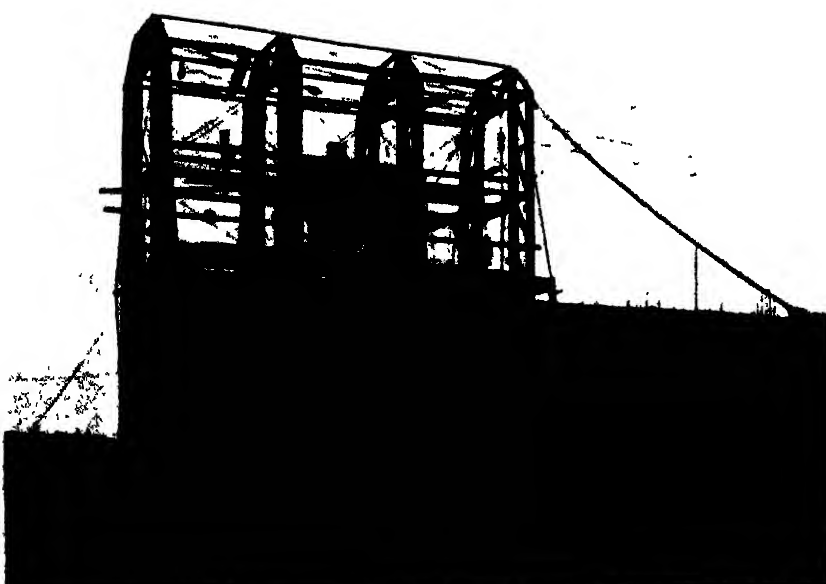


FIG. 130.—TRAVELLER AND FORMS FORMING A SINGLE UNIT: WALL POURED IN TWO LIFTS.

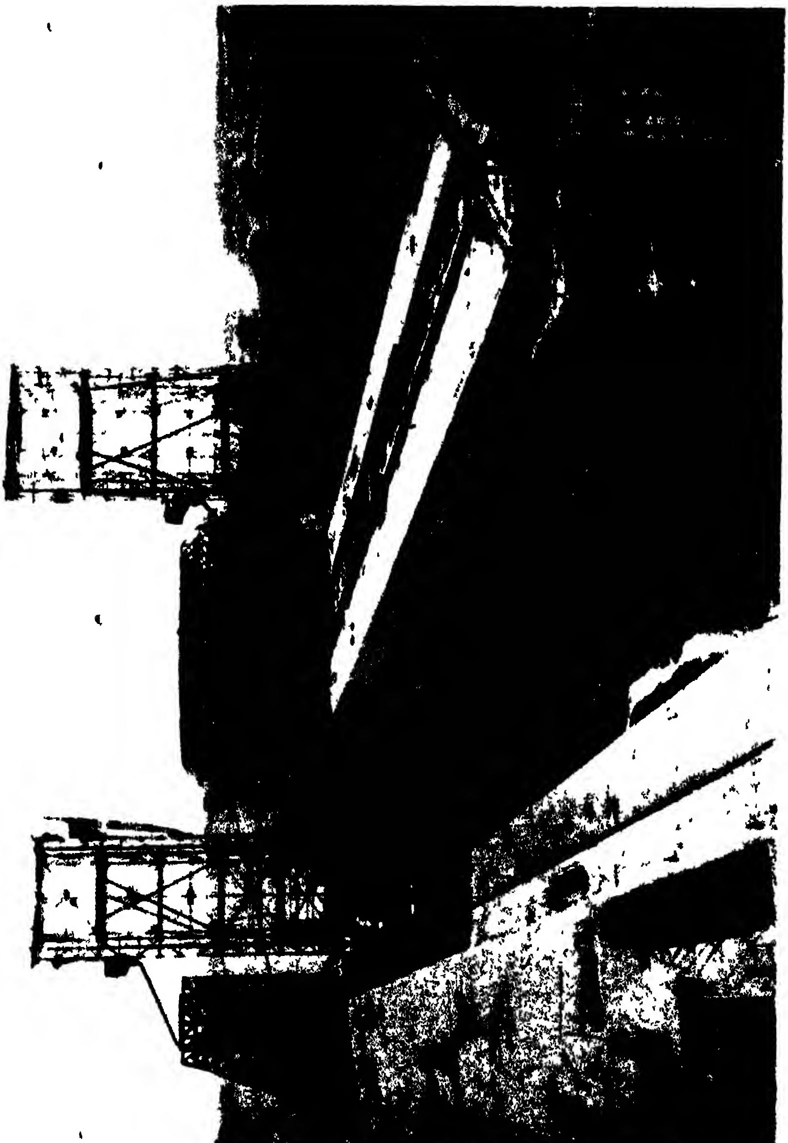


FIG 131.—TRAVELLING WALL FORMS USED IN CONSTRUCTION OF WELLAND CANAL.

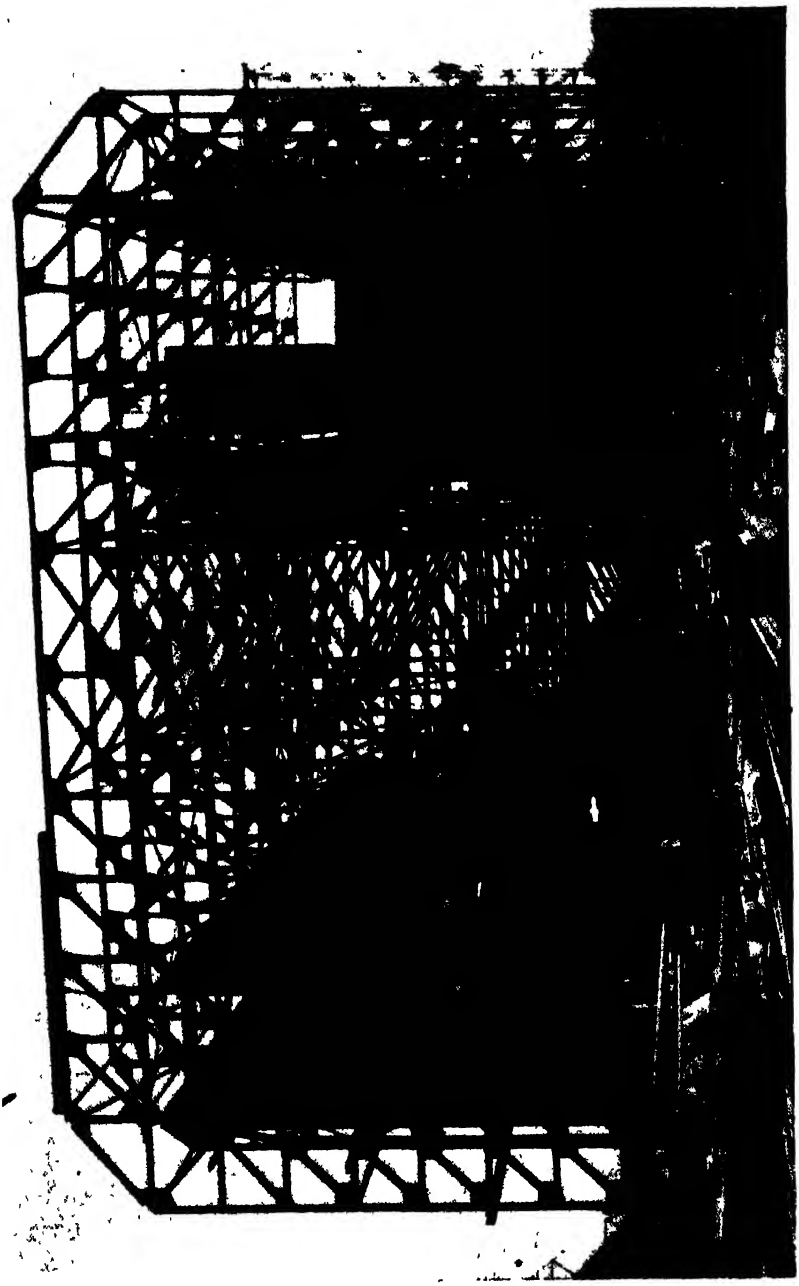


FIG. 132.—DOUBLE TRAVELLING FORM USED IN CONSTRUCTION OF WELLAND CANAL. NOTE SUSPENDED CULVERT FORMS IN RIGHT-HAND WALL.

Fig. 128 shows wall forms for an ore dock. The high traveller is independent and handles eight sets of forms. *Fig.* 129 shows forms used in the construction of a seawall in Florida. One hundred and fifty feet of wall forms and 50 ft. of traveller were used, the forms being self-supporting and the traveller being only used to move them by means of the winch shown. The ratchets are shown hung up out of the way, the traveller being ready to move back on the rails to pick up another section.

In *Fig.* 130 the forms and traveller form a single unit. The wall being very high, it was poured in two lifts as there was possibility of the form being blown over if assembled to its full height The wall being constructed is for a penitentiary.

Figs. 131 and 132 show extremely heavy travelling wall forms used in the construction of the Welland Canal. The main forms are of steel framework with wood lagging. In the right-hand wall shown in *Fig.* 132 are seen two suspended steel culvert forms. These two pictures show that there is no size of wall too large for the use of steel forms, and also show the saving in time and labour by having the forms attached to a traveller so that they can be easily and quickly stripped and moved ahead.

Estimating Cost of Light Wall Forms.

The method of erection is so simple that unskilled labour can handle the forms after a little instruction.

To erect complete and strip 100 sq. ft. of wall forms—contact area—should not require more than six hours of labourers' time. This is for unskilled labour, and should be a maximum. Men experienced in erecting these forms should perform the same amount of work in about half the time, but their rate of pay will be higher.

It will require about the same labour to strip and oil as to erect and wire-up. The same amount of wire ties will be required as for wood forms.

Comparing with the costs given in Chapter VIII, it will be seen that the labour cost on steel wall forms should not be greater than one-third the labour cost of wood forms.

A comparison of the first cost of steel forms with wood forms is more difficult to make. At the present time light steel wall forms will cost 8 to 10 times as much per sq. ft. as will the necessary amount of timber per sq. ft. of wall, but usually less area of forms is required when steel forms are used.

If the steel forms are carried as plant the whole first cost would not be charged to one job, probably not more than 25 per cent.; that is, the cost would be distributed over at least four jobs. Hence, taking into consideration the saving in labour, steel forms compare very favourably in cost with wood forms. Even if the whole cost is charged to a particular job steel forms will often be more economical providing they can be used over a sufficient number of times, so that the saving in labour will be more than the difference in first cost.

CHAPTER XVII

STEEL FORMS FOR CURVED SURFACES.

Conduits and sewers, grain bins and silos, penstocks, subways and tunnels are structures in the construction of which steel forms have almost entirely superseded wood and engineers' specifications now often insist that steel forms be used for these structures

These forms are always fairly difficult to make in wood they can only be used a limited number of times owing to hard handling they have to be repaired frequently, they do not keep their shape so well as steel, and do not give so smooth a finish Also, wood forms are more difficult to strip and move as they require rigid joints and bracing

Structures through which sewage or water flows are required to be smooth to lessen the friction, so that steel forms are particularly suitable for sewers and conduits and result in a large saving in the cost of finishing the concrete Usually the contracts for these structures are so large that

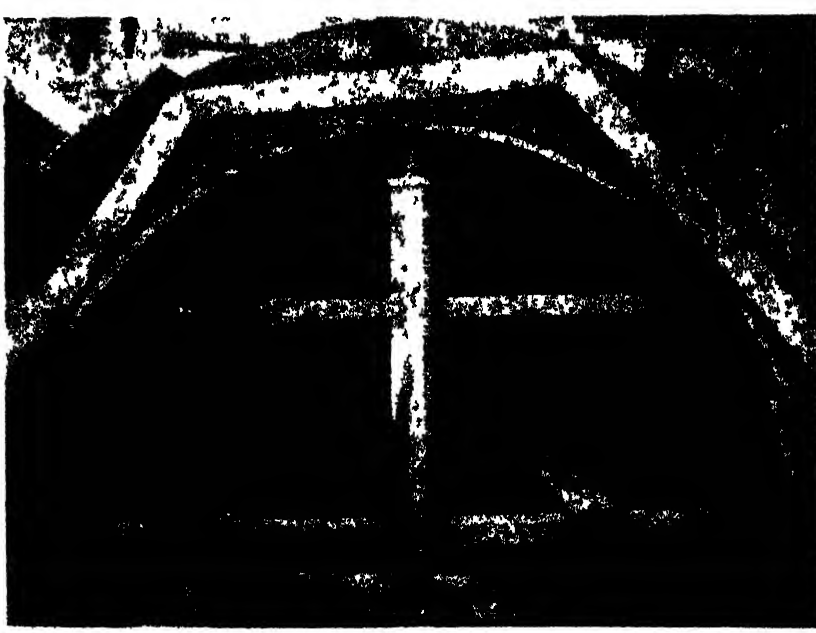


Fig 133 —Half round Sewer Forms with Timber Ties and Screw Jacks for Collapsing

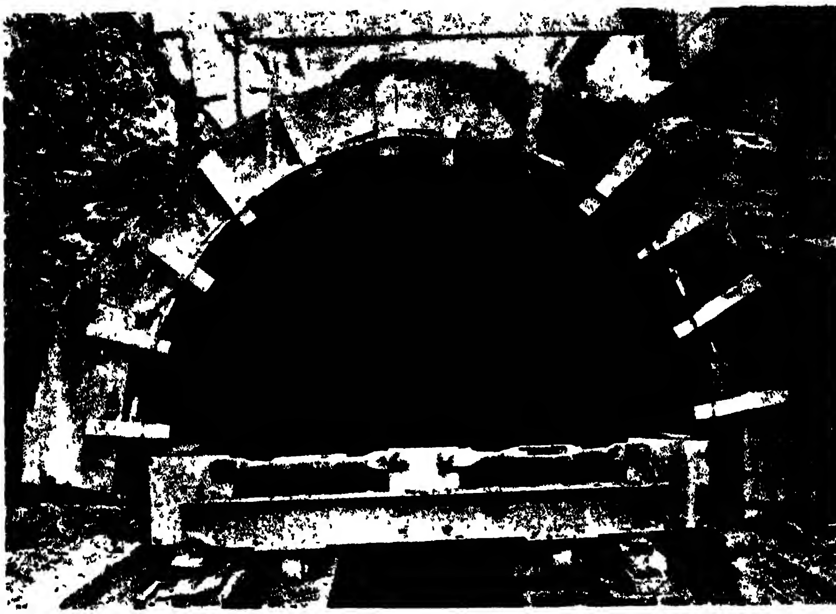


FIG. 134.—A 72-IN. HALF-ROUND SEWER FORM WITH TRAVELLER ATTACHED.

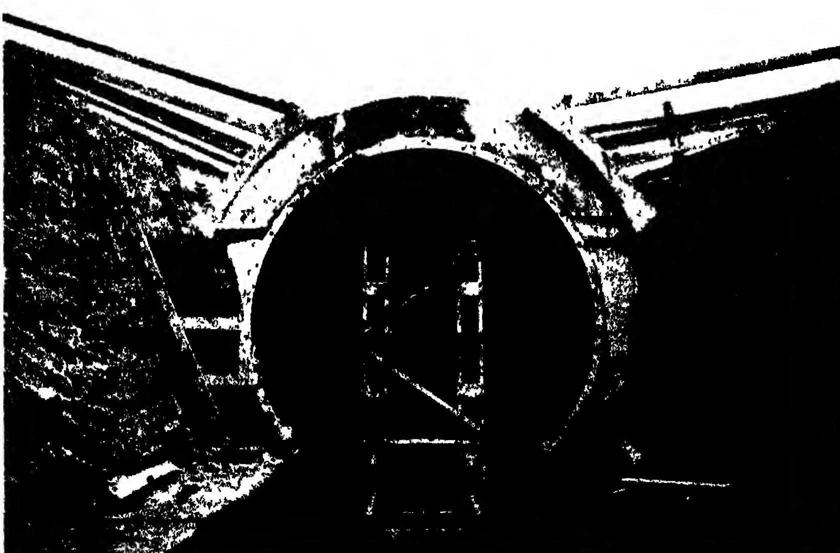


FIG. 135.—CIRCULAR SEWER FORM, 11 FT. 6 IN. DIAMETER, WITH TRAVELLER RUNNING ON TRACKS.

FIG. 136.—A 15-FT DIAMETER SEGMENTAL SEWER FORM WITH TRAVELER.

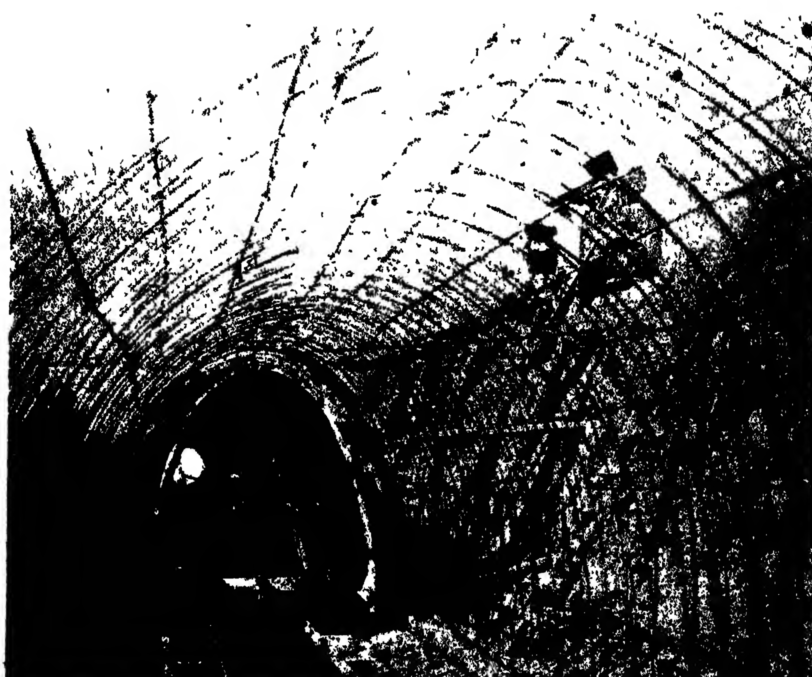


FIG. 137.—HORSE-SHOE SEWER FORM, SHOWN COLLAPSED AND BEING PULLED THROUGH ERECTED FORM.

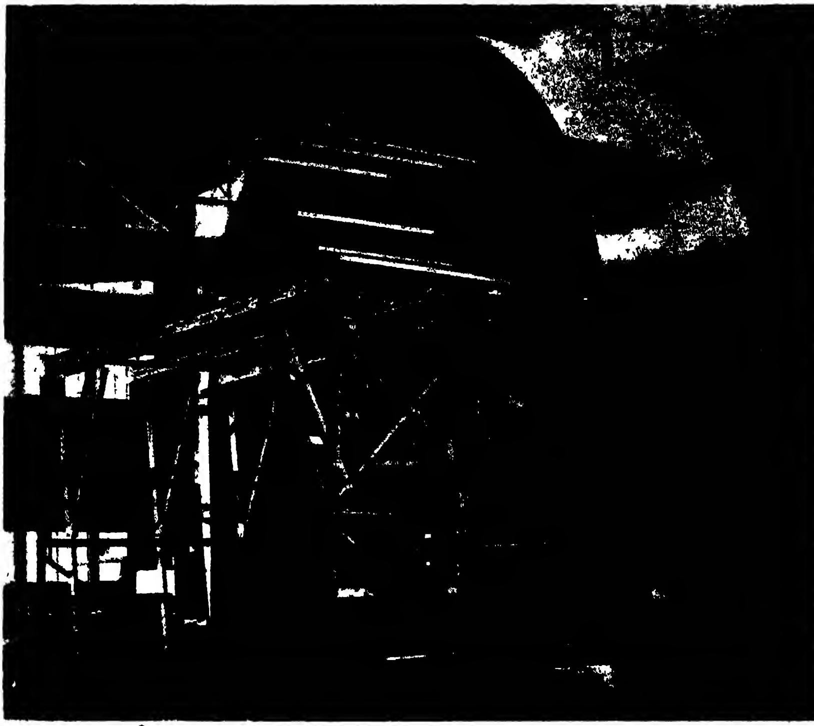


FIG. 138.—TRAVELLING TUNNEL FORM, 32 FT. DIAMETER BY 32 FT. HIGH.

even if the forms have to be specially made they will pay for themselves on one job.

Steel forms will retain their shape; at the same time, owing to their flexibility, they can be easily and quickly collapsed for moving ahead. Another advantage is the absence of bracing, allowing more working space within the form.

In general the forms are built of steel plates and angle-iron stiffeners. Their construction and use are clearly shown in the photographs.

Sewers.—To eliminate friction in the finished sewer the forms are made up with as few joints as possible in the plates. Forms for 15-ft. diameter structures can be built with longitudinal joints only at the crown and springing lines and cross joints 5 ft. apart. There are endless diameters and shapes for sewers; some of the most typical are illustrated.

The simplest form is the circular sewer. To form these a half-round form is generally used, laying the invert first and the crown afterwards. They are made up in 5-ft. sections interlocking together, using as many sections as are required by the speed of the job and facilities for handling the concrete. They are collapsed usually by turnbuckles, which can be seen in the pictures. Another method of collapsing is by means of screw

jacks fastened to one end of timber ties (*Fig.* 133). When used for the invert the forms are suspended from an overhead I-beam track by trolley hangers running on the lower flange of the beam. When used for the crown the smaller sizes have two rollers—short pieces of pipe—on the turnbuckles which run on a plank track when moving, the entire unit moving at one operation.

The larger sizes are equipped with travellers, consisting of channels with rollers attached, running on plank runways laid on the invert (*Fig.* 134). The traveller and form may form one unit or the traveller may be independent, handling several sections which when collapsed move through the forms ahead.

An almost complete circular form for an 11 ft. 6 in. diameter sewer is shown in *Fig.* 135. This is moved by a traveller running on tracks laid on the invert.

Fig. 136 shows the forms for a 15 ft. diameter segmental sewer with the traveller attached. (Note the man operating the turnbuckles by which the form is collapsed, also the steel bulkhead.)

Forms for a horse-shoe shape sewer are shown in *Fig.* 137. This form is in the collapsed condition and is being pulled forward on a track by the attached ropes and block, telescoping through the erected form.

FIG. 139.—TRUSSED TUNNEL FORM IN POSITION FOR CONCRETING.

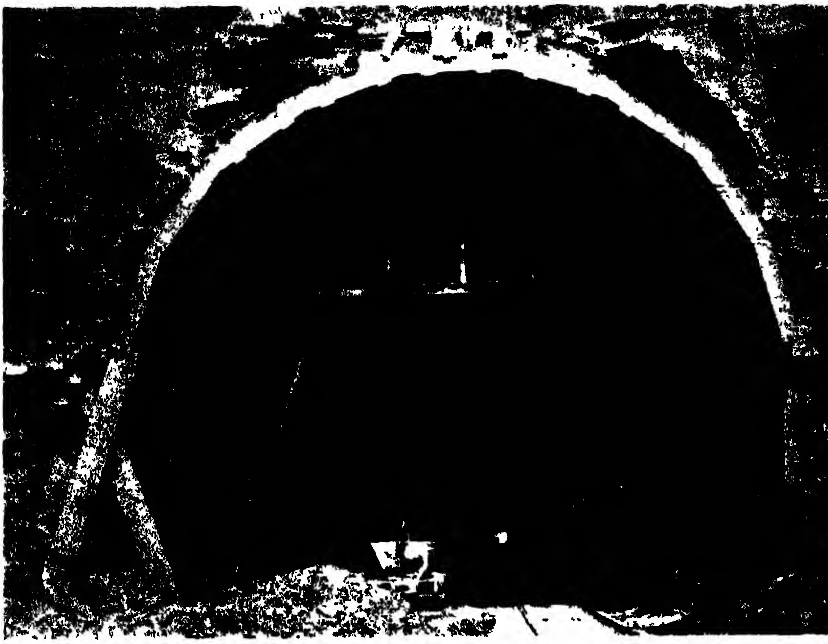


FIG. 140 ARCH RIB TUNNEL FORM, GIVING LARGE WORKING SPACE

It will be seen that very little time is required to set the form, and that concreting may be an almost continuous operation. Contrast the labour required in this method with that required in taking apart the sections of a wood form and re-erecting them.

Tunnels and Subways.—These are some of the largest structures in which steel forms are used. They are particularly suitable and economical for such structures, as they are easy to strip and move ahead, and give a large clear working space, which is important when cars are carrying out the excavated material through the forms.

Fig. 138 shows a typical steel form with traveller for a tunnel 32 ft. diameter by 32 ft. high. The large size of the plates with few joints, giving perfectly smooth surfaces, should be noted. This form was used by the Niagara Falls Power Co.

Fig. 139 shows a tunnel form used in the construction of the Liberty Tunnel at Pittsburgh, U.S A. The form is in place ready for concreting. The whole width of the tunnel is available as working space. The lagging seen at the top of the tunnel is to hold back the earth and rock and is concreted in. This form is of truss design.

Fig. 140 shows a travelling tunnel form of arch rib design, giving a still greater working space.

Fig. 141 illustrates the use of steel forms in building a twin subway.

Penstocks.—These are similar in design to sewers and conduits, but are often much larger in diameter. *Fig.* 142 shows a travelling

penstock form 21 ft. in diameter, used by the Niagara Falls Power Co. It is shown at the mouth of a tunnel. All the forms shown in *Figs.* 133 to 142 were designed and manufactured by the Blaw-Knox Co., by whose courtesy they are reproduced.

Silos and Grain Bins.—All the forms described above travel horizontally. We will now consider forms which move vertically. Tanks are not mentioned because they are usually built with wood sliding forms to avoid construction joints, but they can be built with steel forms in the same manner as silos.

The forms for these structures do not move as a whole, nor are they continuously sliding forms. Instead, the method used is to build the forms of a number of square curved plates interlocking, similar to light wall forms. For large-diameter structures the curvature in the 2-ft. width of a plate will be small, so ordinary straight wall forms may be used in the same manner as for walls. Two or three courses are used, moving one above the other (*Fig.* 143).

For smaller diameters, particularly for silos and grain bins, the plates are attached to radial angle-iron arms at the top and bottom. The radial arms end in collars which slide over a vertical pipe, resting on a casting at the bottom There are holes at intervals of 24 in.

FIG. 141 —TWIN SUBWAY FORMS.

through the pipe, through which short rods can be slipped to help support the collars, which are in two parts bolted together against the pipe.

The "Metaform" method of assembly of the plates and arms is clearly shown in *Fig.* 144. Each unit is self-contained, there being no loose parts. It will be seen that the plates are connected sideways by two clamps which are connected to the plates and vertically by a clamp engaging in an eye in the plate above. The arms rest on two circumferential rings The forms are adjustable to different diameters by inserting similar plates of different widths, and the arms telescope to

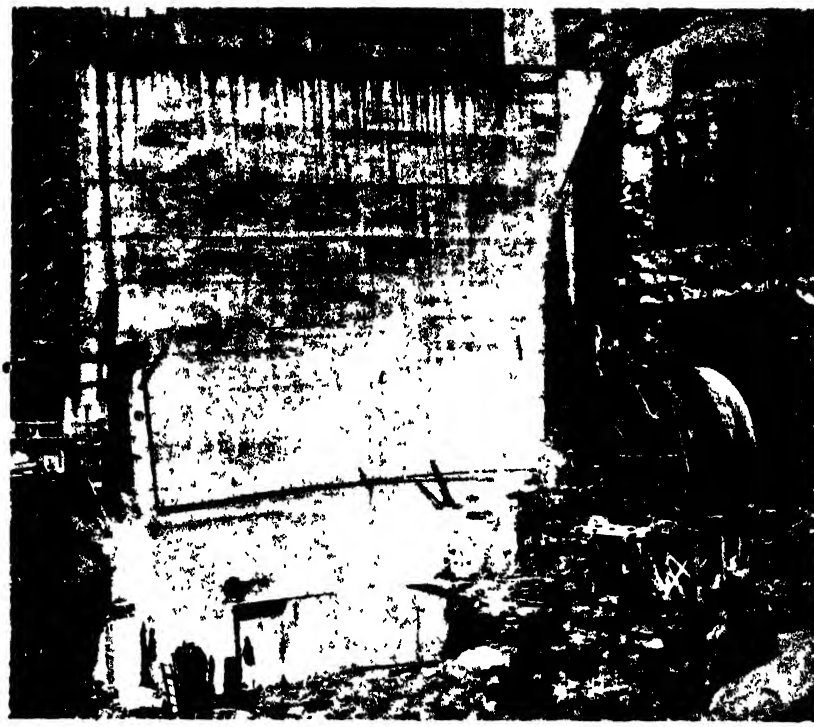


FIG. 142.—A 21-FT. DIAMETER PENSTOCK FORM.

suit different diameters. Spacers slipped over the tops of the plates hold the inner and outer form the correct distance apart. Scaffold planks are thrown over the upper arms to form a working platform.

For ordinary silos or bins two courses are used, pouring 4 ft. the first day and 2 ft. each succeeding day, stripping in the morning and pouring in the afternoon. If there is a battery of bins, better time can be made with three courses pouring 4 ft. a day. The outside panels are removed first and set in place on top of the upper course; then the reinforcing steel is placed; and, lastly, the inside form. The outside panels are handled one by one from the top with a long hook which loosens the clamps, engages the eyes of the top clamps, and lifts the panels up to the

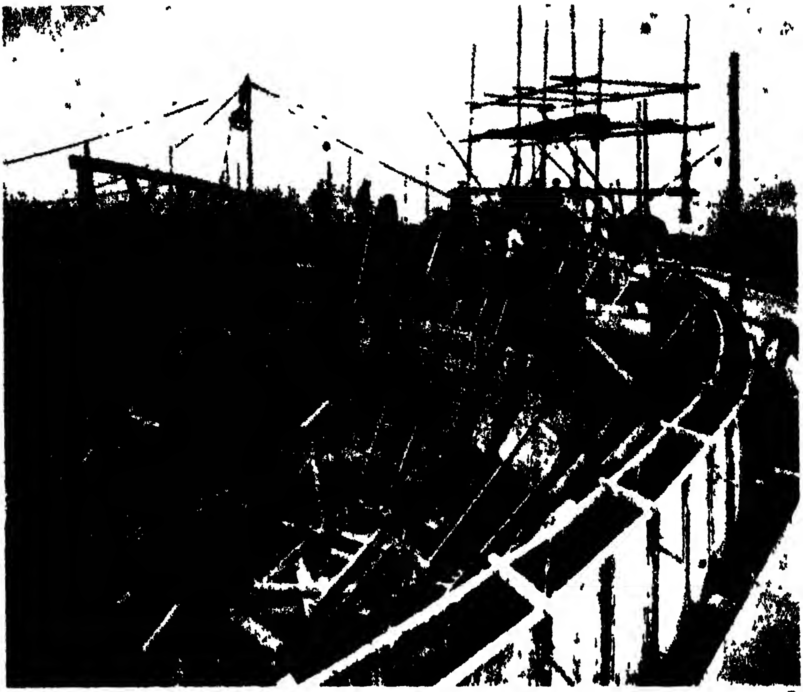


FIG. 143. METAL FORMS FOR CIRCULAR STRUCTURES

FIG. 144.—"METAFORM" METHOD OF ASSEMBLING CIRCULAR FORMS FOR SILOS AND GRAIN BINS.

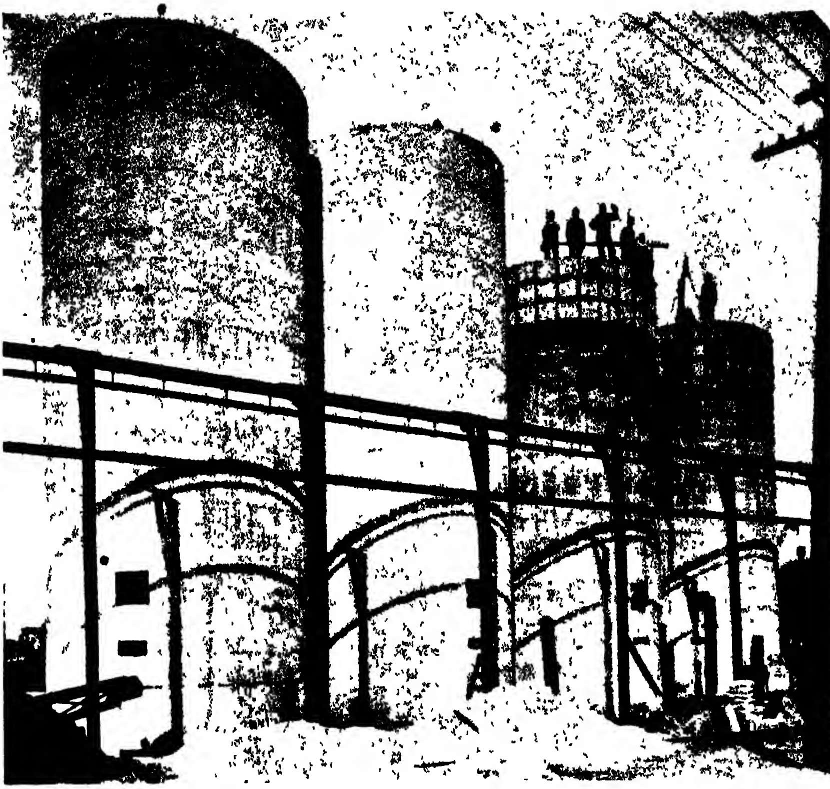


FIG. 145.—TWO THREE-COURSE OUTFITS OF "METAFORMS," SHOWING HOIST AND DISTRIBUTOR.

FIG. 146.—GRAIN BINS BEING BUILT UP TOGETHER WITH INTERLOCKING STEEL FORMS.

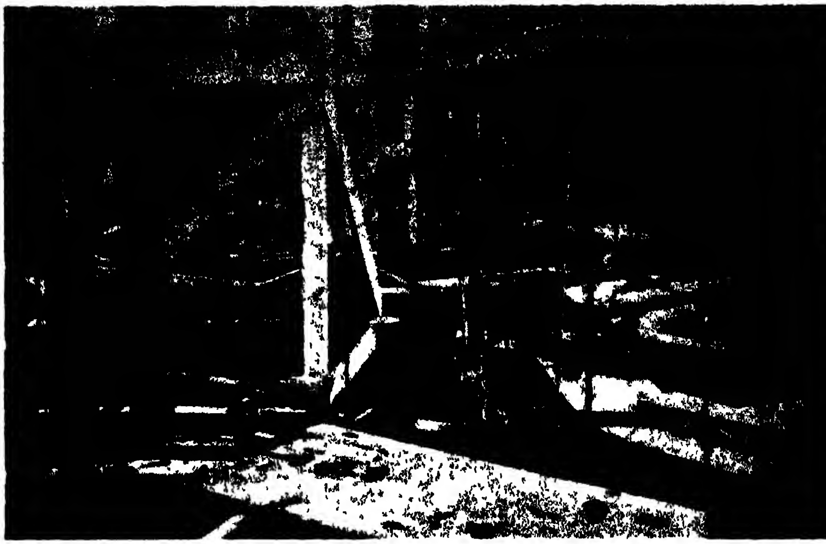


FIG. 147 — FORMS AT CONNECTIONS OF BINS ILLUSTRATED ON P. 222

course above. The inside panels and the arms are unclamped and raised by hand.

Fig. 145 shows two three-course outfits in use, and *Figs.* 146 and 147 various stages in the construction of a battery of four grain bins, the bins being carried up together. As pouring thin walls at a great height is always troublesome, the Metal Forms Corporation provide with their silo forms a hoisting attachment, which is a light steel derrick supported by the pipe mast and the forms, and a distributor which is a tip bucket running on a circular track near the inside form. Hoist and distributor can be seen at the top of the bins in *Fig.* 145. A similar bucket is used for hoisting the concrete, so that one bucket is being emptied while the other is being filled.

For combinations of bins special connecting pieces are made (*Fig.* 147). Steel door, chute, and roof forms are made for use in silo construction.

Cost.—The cost of placing and stripping circular steel forms is about the same as given for light wall forms, the operations being similar.

CHAPTER XVIII.

ARCH FALSEWORK.

There is no part in the construction of a concrete arch bridge that is more important than the centering. On its design and construction largely depends the success of the job. The forms for the abutments, piers and spandrel walls present no particular difficulty, but to the methods used in supporting the arch ring there usually has to be given—or should be given—considerable thought.

There are so many different designs of bridges and so many different conditions under which they have to be built that a whole book would be necessary to treat the subject thoroughly, but in this chapter are indicated the different conditions commonly met with, which will govern the type of centering to use, and a few typical designs which can be used in the majority of cases are given

Difficult centres for special conditions should be designed by engineers experienced in this kind of work, with both a theoretical and practical knowledge. An arch centre is a temporary structure and should be designed as light as is consistent with strength and rigidity.

When possible the simplest design, that is, the design most economical in labour, should be chosen, since labour efficiency is the uncertain item while timber cost can be calculated fairly closely. The amount of timber required for two different designs for a centre may be about the same, but the labour cost of erecting and stripping one design may be twice that of the other.

The different types of arches are solid-barrel, open-spandrel ribs, closed-spandrel ribs, and bowstring girders.

In solid-barrel arches the load is distributed uniformly over the whole width of the arch and hence over the centering.

In the open-spandrel ribbed arch the load is generally concentrated in two ribs so that the main centering is under the ribs.

In the closed-spandrel arch, or an arch in which the ribs are carried up to support the roadway, the load is still more concentrated in narrow ribs, probably five or six.

Bowstring girder arches are a separate consideration because the form-work required for the deck and ribs is similar to the forms for beam and girder buildings.

The kind of arch does not affect the type of centering design particularly, but only the distribution of the supports, which are either uniformly spaced across the arch or concentrated at the ribs. The type to use will

depend on the span and rise of the arch, condition of the foundation, depth of water, liability of floods, distance between supports required for passage of traffic, ice and débris, and the sizes and lengths of timber available. In the average bridge, foundation conditions are the most important.

The necessary features of a good design are that the combined costs of labour and material will be a minimum, that there will be the greatest salvage of material, that the forms will be sufficiently strong and stiff, that they are not changeable in level and shape, and that they can be stripped easily without shock to the concrete.

Types of Centering.

The types of centres that can be used are trusses, bents, and steel centres, shown in diagram form in *Fig.* 148, omitting braces and only showing the main members.

Trussed centres are only used when no other type is possible, and this does not often occur. They are, however, sometimes required over deep ravines or deep water or when a clear passage-way must be left for traffic. They are expensive to frame, there is little salvage of material, and they are apt to deform badly under load. They should have one or two intermediate bent supports, when possible, to cut down the deflection.

They have to be designed carefully and built accurately, and should only be designed by an engineer. Investigation of stresses should be made for the different loading that will occur under the proposed method of building the arch. Some members will be subject to reversal of stress, and this must not be overlooked. Stiffness is important, and probable deflection of the truss should be calculated and provided for.

The bowstring truss with Warren bracing is the type generally used and is the most satisfactory (*Fig.* 148, *e* and *h*). At *e*, two centre bent supports are shown; if they cannot be used the truss must be made sufficiently stiff of itself not to deflect seriously under load. At *h* the span of the truss is reduced by supporting it at each end from the bents at the piers; or, omitting the bents, from the piers themselves. The depth of the truss in this case could have been made equal to the rise of the arch, but long diagonal members should be avoided by raising the lower chord.

The Howe truss is sometimes used, supporting bents at the panel points as shown at *g*. The design of these trusses is beyond the scope of this book. Some data and tables for their design are given in Trautwine's "Civil Engineer's Pocket-Book."

The bent design (*Fig.* 148, *a* to *d*) should be used whenever possible, and there are a sufficient number of variations in design to make it suitable for almost any condition. At *f* is shown a type of bent design occasionally used under special conditions. Steel centering, shown at *i*, is now being largely used whenever there are a number of similar spans, or in high bridges to avoid using designs such as shown at *e* to *h*.

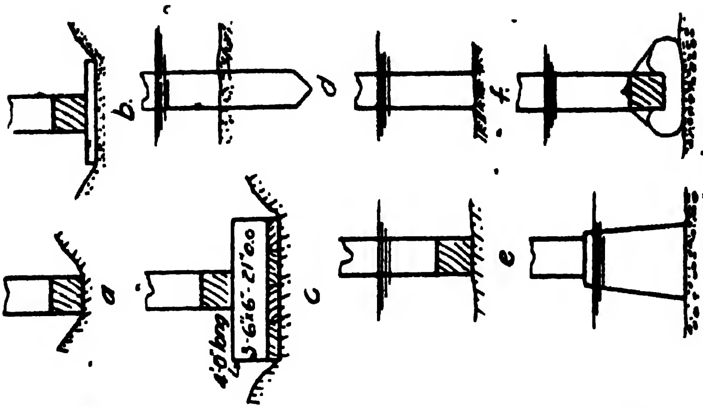


Fig.149. Types of Post Foundations

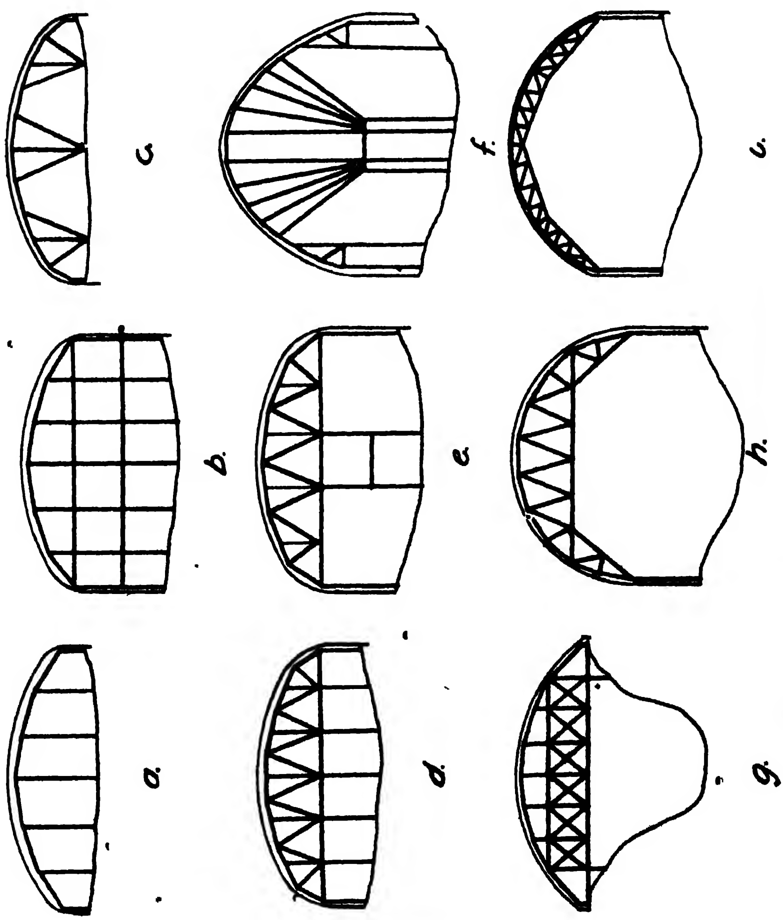


Fig.148. Types of Centering.

Bent Centering.

By "bent centering" is meant the supporting of the arch by posts from the ground. This type will cost least, deform least, give the greatest salvage of material, and is the simplest to build. The posts may all be vertical, which is the best arrangement when possible. For low bridges the posts may be single timbers reaching from the foundation to the arch, as at *a*, or may be built up in tiers as shown at *b* for high bridges.

When it is desired to cut down the number of foundations the posts may be inclined, as at *c*. When it is desired to reduce the span of the stringers without increasing the number of vertical posts, a combination of *b* and *c* may be used, as shown at *d* In very high arches, to avoid carrying all the posts to the foundation, they are sometimes arranged fan-shape, approximately at right angles to the soffit of the arch and supported on a centre pier made up of bents, as at *f*. This method will give more clearance for the passage of traffic.

Combinations of bents and trusses can be used to suit particular conditions. The vertical post bent centering will be considered the standard design to use, combining it with the inclined post type when necessary.

Having decided that the vertical post bents can be used, after studying local conditions, the method of supporting the posts must be determined, governed by the depth of water and character of foundation. It is better to do this before designing the centering, as conditions may affect the design.

Foundations.

Foundations must be examined and tested in order to estimate the probable bearing value of the soil. When there is no water, or only a small amount, and the foundation is good, the posts can be supported directly on timber sills, as at *a*, *Fig.* 149. When more bearing area is required, mud sills of 2 in. or 3 in. plank can be added as at *b*. If the posts are close together it is generally better to have a continuous sill under the whole bent, but if they are far apart individual sills as at *c* will distribute the load more uniformly. Timbers used in the sills must be heavy enough to distribute the load without bending, or settlement will occur.

Top soil should be removed until firm ground is obtained before setting the sills. The effect of rain on the ground should be noticed, as a foundation good in dry weather may be poor when wet. Concrete sills are often advisable when there is danger of settlement and are useful to level up uneven ground.

If the river bottom is soft and incapable of supporting much load, it will be necessary to drive piles, as at *d*. They will generally be required with sandy bottoms, and must be driven until they will carry the desired load, using one of the pile formulas in common use. They should be cut off as low as possible in deep water to avoid a long length of unsupported

pile. Sometimes, however, they can be cut off high enough to obtain effective bracing between the water level and the caps. In most cases the springing elevation of the arch is about the right level for cut-off.

If the river bottom is hardpan, rock, compacted gravel, or any other hard material, and the water is not over 5 ft. or 6 ft. deep, the posts may be set directly on the bottom with sills attached, as at *e*. Sills may be omitted if the foundation is hard rock.

If there is a foot or two of soft silt or mud overlying rock, the posts can be driven to the rock by hand, the ends being square, as at *f*.

If the foundation is too hard to drive piles economically, and yet the top is too compressible to use timber sills, which is often the case with a gravel bottom, it is best to use concrete footings either individual, or preferably continuous, under a bent, as at *g*. This cannot be done economically if the water averages over 4 ft. deep, in which case a good sill can be made by spiking a short heavy timber to the bottom of the post forming a T and tying to this cross-piece a bag—or two bags if necessary—filled with dry concrete, as at *h*. The concrete will adapt itself to an irregular bottom, giving an even bearing, and will hold down the post against floating. The author has successfully used this method in 9 ft. of water. When the water is over 10 ft. deep and piles can be driven, that is the safest method to use. The length of pile unsupported above the river bed should not be much over 20 ft. and its strength as a post should be investigated.

If piles cannot be driven the foundation problem becomes difficult. If may be solved by building one or two centre piers of weighted cribwork or bents, similar to *e* and *f*, *Fig.* 148. The only alternative is to use a truss design, preferably of steel, which can be done economically if there are several spans. Several methods may have to be used in the same bridge, if the foundation is not uniform.

When placing posts with sills attached in deep water, particular care must be taken that the sills are resting firmly on the bottom and not on the edge of a stone or hole.

Detail Design.

It is best to decide first what size timbers and lengths it is desired to use, the comparative prices, and delivery that can be made. This will save possible trouble and delay after the design is made.

Arch centering is designed for the dead load of the arch ring only, with possibly an allowance for live load, depending on how the arch is poured. With very high centering it may be necessary to take its own weight into account. The various parts to design are the lagging, stringers, caps, and posts.

The dead weight of the arch ring will increase from the crown to the springing, and will act vertically, and since the stringers will always be inclined to the vertical except at the crown they will not have to carry the full dead weight. For calculation purposes the arch ring is supposed to be divided into longitudinal strips 1 ft. wide.

To find simply the loads acting on the centering, divide a cross-section of the arch by radial lines into a number of blocks of equal length about 8 ft. to 10 ft. long (*Fig.* 150). At the centre of each radial line draw a vertical representing to scale the unit weight of the arch at each point, which will be the radial depth multiplied by 12 (approx.). From the ends of these verticals draw lines at right-angles to the radial lines, forming triangles *oab*. Then the side *ob* will represent to scale the unit load *N* acting radially to the soffit of the arch, for which the centering must be designed. The sides *ab* or *oc* represent the forces *V*, which are partly resisted by the friction between the concrete and the lagging, and partly by struts if the concrete is poured in transverse sections, or by the

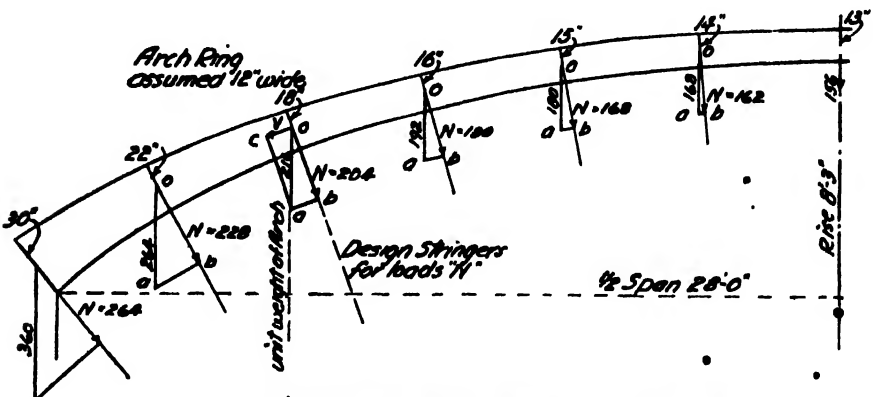


Fig. 150. Method of finding Load on Centering.

concrete already poured if poured in longitudinal sections, the force being transmitted to the abutments. These forces *V* are neglected in the design of the centering.

It can now be seen how the design load varies along the arch. It will be noticed that the steeper the curve the smaller the proportion of the load carried by the centering. The lagging is designed for the maximum value of *N*, which will be near the springing generally. Dividing *N* by 12 will give the equivalent thickness of slab, and the maximum span of the sheathing can be obtained from Table 1, or the formula if it is outside the range of the table. If no live load need be considered the allowance of 75 lbs. per sq. ft. in Table 1 can be turned into the equivalent of 6 in. of concrete and this amount added to the thickness of the slab; that is, an 18-in. slab with no live load will give the same maximum span of sheathing as will a 12-in. slab with 75 lbs. live load.

The maximum span of the lagging for the minimum value of *N* can also be found. One method would be to vary the spacing of the stringers uniformly from a minimum at the springing to a maximum at the crown. It will be seen from Table 1, however, that a large difference in load means only a slight change in the span, which is not worth while since it is more

difficult and confusing to lay out and cannot be done uniformly as the stringers must lap. Instead, it is better to space the stringers uniformly throughout and take care of the decreasing load by varying the spans. The loads N multiplied by the spacing of the stringers in feet will give the variation of load per lineal foot of stringer.

Taking an average load near the springing and near the crown, the minimum and maximum spans for a certain size stringer—measured along the slope—can be found from formulas 1 and 2 (Chapter III), checking the size for shear and deflection by formulas 8 and 10.

The size of timber should be such that the spans will be between 8 ft. and 14 ft. The spans must then vary between these limits, according to the variation of the loading, being generally fairly constant from the crown to the quarter-point and decreasing more rapidly from there to the springing. One or two trials will determine the best spacing of the bents, when the size of the stringers should be checked, using the actual loads. These are obtained by taking an average of the loads at each end and assuming it is uniform over the stringer—this is only approximate but sufficiently close.

The loads on the joists are transferred to the caps, which can be designed by the formulas for concentrated loads in Chapter III.

Allowable shearing stress will often govern the size of the cap, as the loads are heavy and the spans short. The caps generally span between 4 ft. and 7 ft., which will be the spacing of the posts transversely in the bent. To reduce the shear it is best to place the posts directly under stringers, generally carrying two stringers on the span.

Knowing the load on the posts, their size is calculated by the formula for posts in Chapter III.

The bearing stress at the points of contact of the timbers should be investigated. The stringers will rest obliquely on the caps and must be notched out to give a sufficient square bearing area. The posts must be sufficiently large to give a safe bearing stress on the cap and on the sills below; iron plates can often be used to reduce the stress.

There are two other forces acting on the centering to be provided for, but which need not be calculated except in special cases of very high bridges.

The first is the horizontal thrust on the caps due to the inclination of the stringers. This is taken care of by notching the stringers over and wedging against the caps, by continuous longitudinal bracing just below the caps and by inclined struts against the caps near the springing (*Fig.* 154).

The other force is wind pressure, which is resisted by adequate sway bracing of the bents. Longitudinal cross-bracing is necessary to stiffen the structure, and is more important near the top than the bottom of the centering. Centres should be cross-braced with longitudinal and diagonal braces about every 12 ft. in height. Longitudinally, the diagonal braces may be omitted in every other bay near the bottom.

Whether the arch is a solid barrel or consists of ribs the centering is

designed in the same way, though in the latter case the posts will be closer together under the ribs.

Stresses and Timbers.

If the lagging is to be used not more than two or three times, 1 in. tongued and grooved pine is satisfactory. If used several times, 1½ in. spruce will stand the wear and tear better. Sometimes lagging 2 in. to 4 in. thick is used, spacing the stringers farther apart and carrying them directly on the posts, omitting the caps.

Stringers, caps, sills, and posts should be of the best timber available, long-leaf yellow pine being the first choice and then Douglas fir or equivalent timber. For bracing, spruce or fir is the best, though short-leaf yellow pine and sometimes hemlock can be used for less important structures.

Lengths over 18 ft. to 20 ft. are expensive and difficult to handle; it is better to so design the centering that lengths of 12 ft. to 16 ft. can be used. For a high centering the author prefers to build it up in tiers about 12 ft. high rather than to use long posts, since the timber costs less and it is quicker and easier to erect (*see Fig.* 154). An exception to this is when rough unsawn timber is used for the posts.

The safe stresses given in Chapters II and III can be increased by 50 per cent. when long-leaf yellow pine, Douglas fir, or equivalent timber is used. This will give the safe fibre stress in bending of 1,800 to 2,000 lbs. per sq. in.; in shear, 300 lbs.; in bearing, 600 lbs.; and for posts about 1,200 lbs. per sq. in.

The best sizes of timber to use are 2 in. by 10 in. to 3 in. by 12 in. for stringers; 6 in. by 6 in to 8 in. by 12 in. for caps and sills; 4 in. by 4 in. to 8 in. by 8 in. for posts; and 2 in. by 6 in to 3 in. by 8 in. for braces.

Typical Designs.

The following designs are selected from the author's experience as being typical centres for the different kinds of concrete arches, and which were built under various foundation conditions such as are commonly encountered.

Fig. 151 shows the design of the centering used for an open spandrel-arch bridge of two 86 ft. arches, each arch consisting of two parabolic ribs 5 ft. wide and 18 in. to 5 ft. deep. The water was 3 ft. to 6 ft. deep, the current was swift, and the bottom not hard enough for sills—so pile bents were used. As this was comparatively the most expensive part of the centering as few piles as possible were driven (two to a bent under each rib). Using so few piles all braces were made a little heavier than would ordinarily be used. The piles were 25 ft. long, cut off at the springing level, and the bents were erected in two tiers 12 ft. high on top of the pile caps. Wedges were placed under the top tier of posts. *Figs.* 152 and 153 show two construction views of this centering.

A centering for a similar type of bridge, but of much greater span and rise, is shown in *Fig.* 154. The span is 200 ft. and the rise from bed of

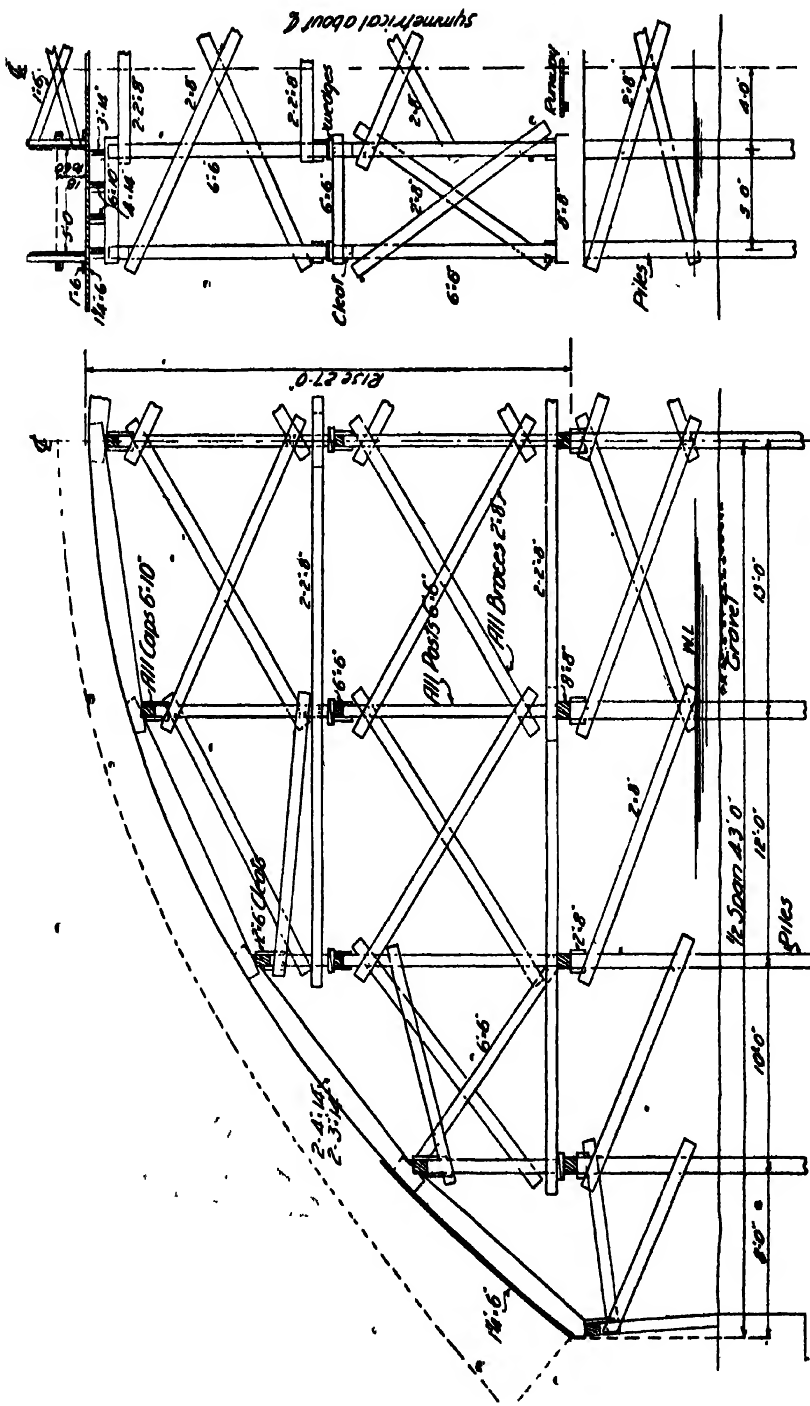


Fig. Desi for 86' Sp. Two-Rib Open Spandrel Ar

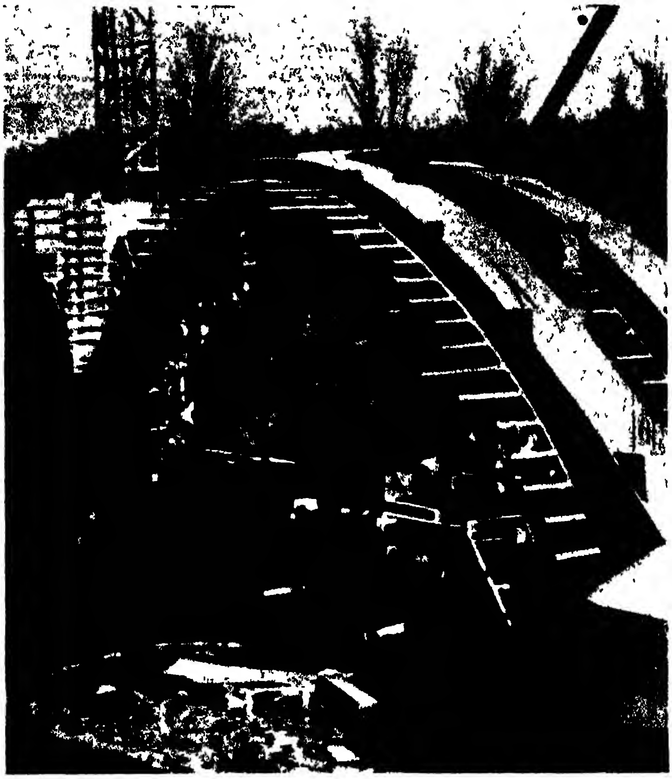
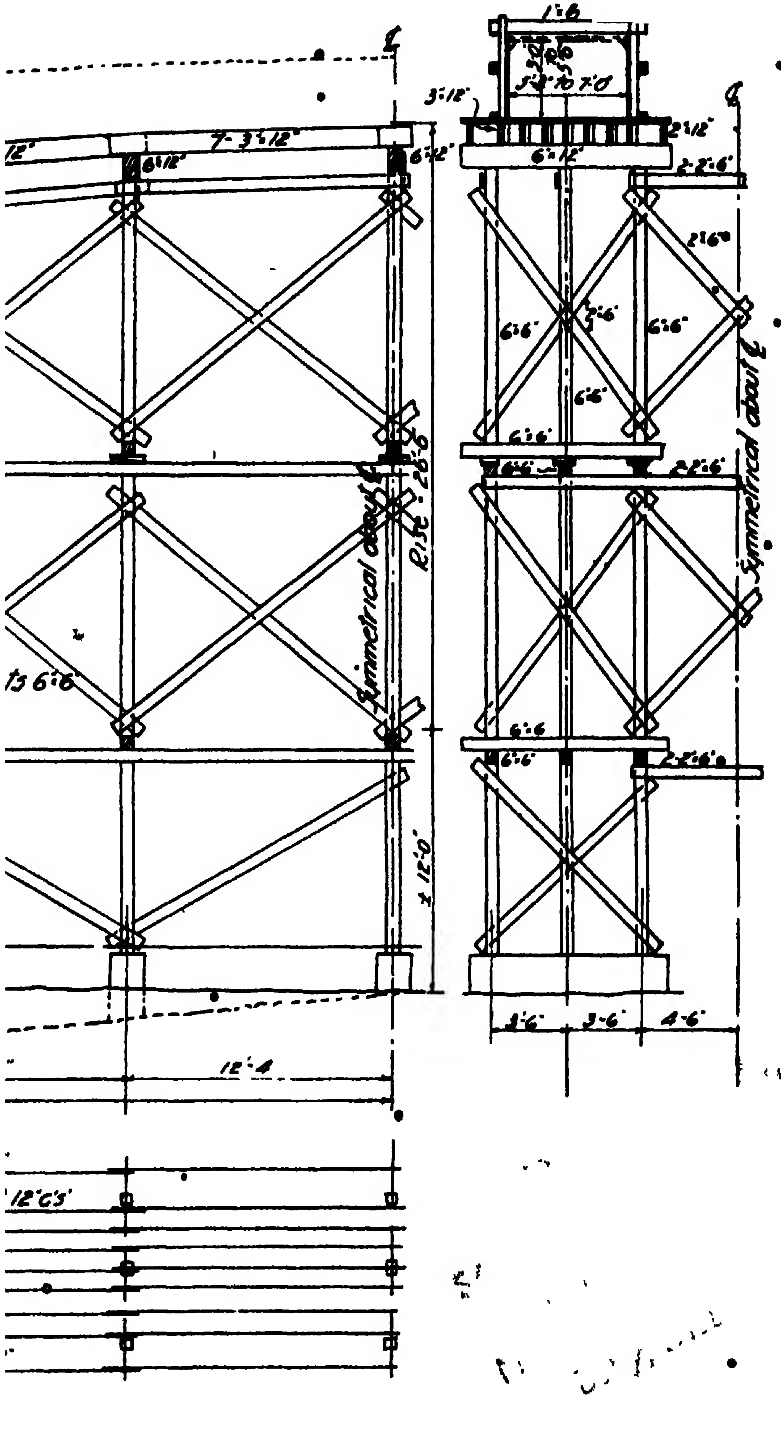


FIG 152.—PILE BENT CENTERING FOR 86 FT SPAN 2 RIB ARCH FIRST ARCH COMPLETED

FIG. 153.—SECOND ARCH BEING POURED. WEIGHTED CABLEWAYS AROUND RIBS OF FIRST ARCH ALLOW CENTERING TO BE STRIPPED FOR USE ON SECOND ARCH.

FIG. 155.—3-TIER VERTICAL POST BENT TYPE OF CENTERING FOR A TWO-RIB ARCH OF 200 FT. SPAN AND 40 FT. RISE.

FIG. 154.


1'-6
3'-12"
7'-3"x12"
6"x12"
2"x12"
2-2"x6"
2"x6"
6"x6"
Symmetrical about ℄
Rise 26'-6"
12'-0"
ts 6"x6"
3'-6"
4'-6"
12'-4"
12"C's

river to crown of arch 40 ft. Each of the two ribs varies in width from 5 ft. 4 in. to 7 ft., and in depth from 3 ft. to 5 ft. 6 in. The depth of water varied from a few inches to 6 ft., and had a rise and fall of 2 ft. each day. The river bottom consisted of 8 ft. of gravel overlying rock, the top of the gravel being loose and compressible with the fine material washed

FIG. 156.—VIEW OF CENTERING, SHOWING DETAILS NEAR ABUTMENTS.

out. This gravel was too hard to drive piles into economically, and as the loads on the posts were high (10 to 13 tons) it was thought there would be danger of settlement with timber sills. After making a test of the bearing value of the soil (which was found to be about 1½ tons per sq. ft.) without appreciable settlement, it was decided to build concrete piers continuous under the three post bents supporting each rib. The width of the piers

was increased in deep water to prevent possible overturning. The concrete was mixed 1 : 3 : 6, with gravel dug out of the river bed near the piers. The posts were set directly on the concrete. The bents were built up in three tiers 12 ft. high. The longitudinal 6 in. by 6 in. sills stiffened the structure, formed a working platform and in particular took the horizontal thrusts on the four lower caps on each side. (Note how these thrusts are transmitted to the sills through 6 in. by 6 in. inclined struts, shown also in *Fig.* 156.) *Fig.* 155 shows a view of this centering when completed.

An alternative design for this centering, shown in *Fig.* 157, was made, using long inclined posts. This is typical for an inclined post design. The main object of this design is to reduce the number of footings by concentrating three posts at one footing. In this case the number was reduced from 17 to 10. To shorten the lengths of the posts they start at the springing line, vertical bents being carried up to that elevation. The bents are connected longitudinally by two rows of 6 in. by 6 in. timbers over each post, with the wedges between, the timbers butting tight against the end bents. Over each wedge is a short 4 in. by 6 in. to give more room for driving. If the wedges in this case were placed at right-angles they would have to be very wide to cover the three posts. Vertical posts are cleated to the caps, and the diagonal posts have T-shape supports for the caps, which are placed radially to the soffit. The two lower caps are strutted diagonally, as shown. Each post is braced longitudinally with two rows of braces in pairs. This is necessary to prevent sag and bending when using inclined posts. Cross-bracing on vertical posts connects the bents under both ribs; on inclined posts cross-bracing is only over the three posts of one bent. The details of stringers and lagging are the same as in *Fig.* 154.

The objections to this type of design are the long lengths of posts required and the higher labour cost, which is much higher than when all vertical posts are used. The inclined posts have to be cut exactly to fit and are more difficult to erect. With this type, levels and measurements have to be very carefully made, as the posts have to fit exactly between two points; if they do not it is difficult to change them. The main advantage is that fewer post footings are necessary.

A centering design for a 66 ft. span, one of a six-span bridge with six ribs of the closed spandrel type, is shown in *Fig.* 158. In this bridge the ribs extend up to and directly carry the roadway. There is no water, and the posts set directly on concrete footings as the loads are high and the posts too far apart for good distribution with continuous sills. The loads on the stringers being high, inclined posts are used to reduce the spans, which also reduces the number of footings required. The ends of the posts rest on short 4 in. by 6 in.'s, and are well tied together with two longitudinal braces, cleated across the top to prevent spreading. These braces should be tight against the sides of the piers, so that they can act as struts to take any possible thrust from the inclined posts. To act as ties, one of the bottom boards of the ribs is carried across the arch in

FIG. 157.

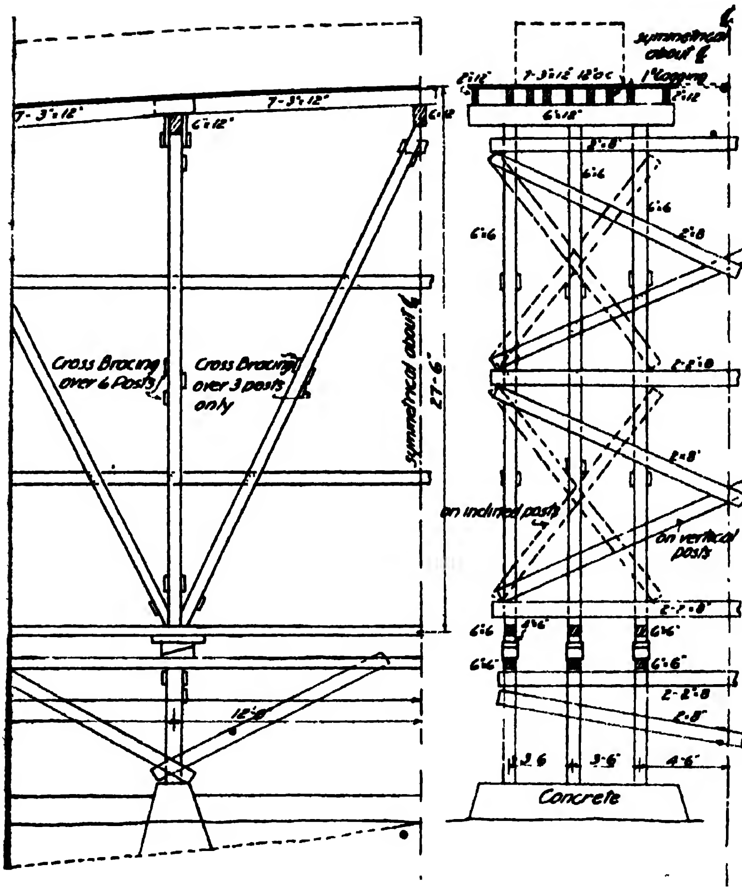
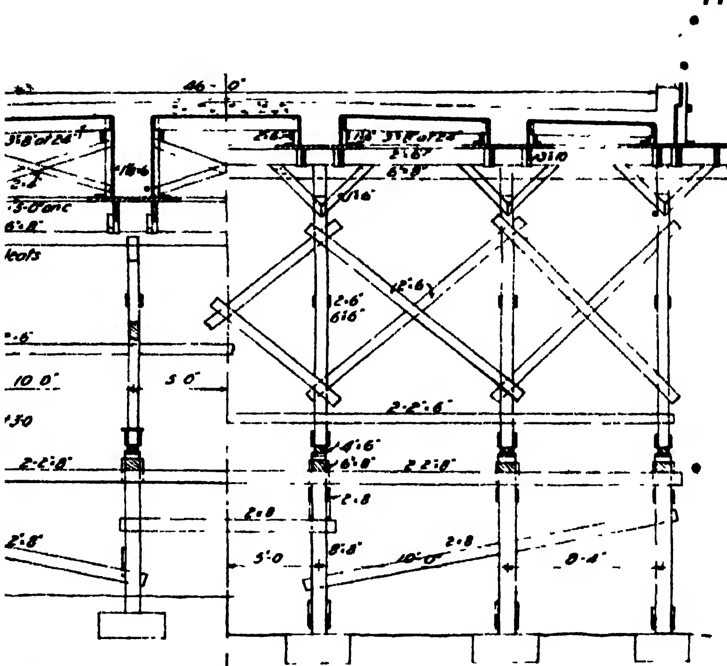

7-3"x12"
7-3"x12"
6"x12"
Cross Bracing over 6 Posts
Cross Bracing over 3 posts only
symmetrical about ℄
27'-6"
12'-0"
symmetrical about ℄
7-3"x12" 12"o.c.
1" Logging
6"x18"
2"x8"
6"x6"
2-2"x8"
on inclined posts
on vertical posts
3'-6"
4'-6"
Concrete

FIG. 158.



TION AA

SECTION BB

long lengths every 5 ft., the other boards being short. Curved strips are nailed to the stringers instead of sawing 3 in. by 12 in. timbers. The sides

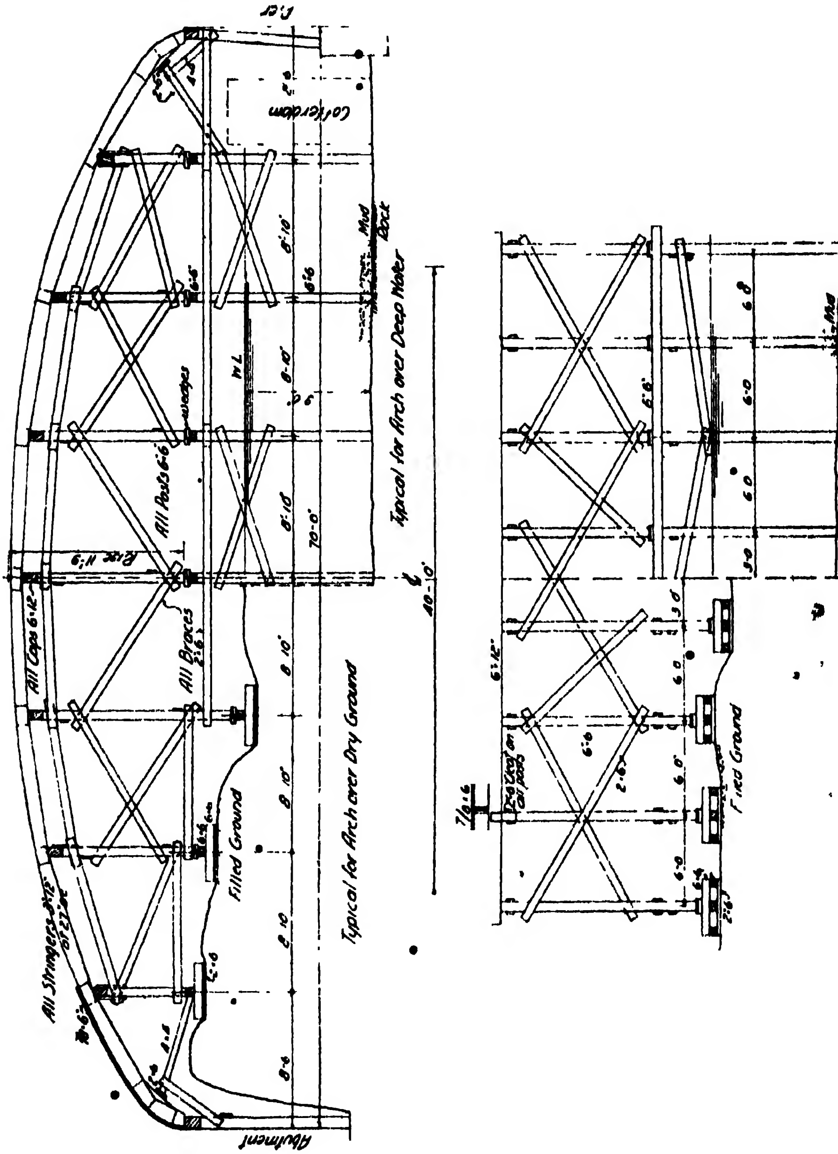


FIG 159 —DESIGN FOR SOLID-BARREL ARCH—3 SPANS OF 70 FT.

of the ribs are formed like walls or deep beam sides, and are braced across between ribs. Other details are clearly shown.

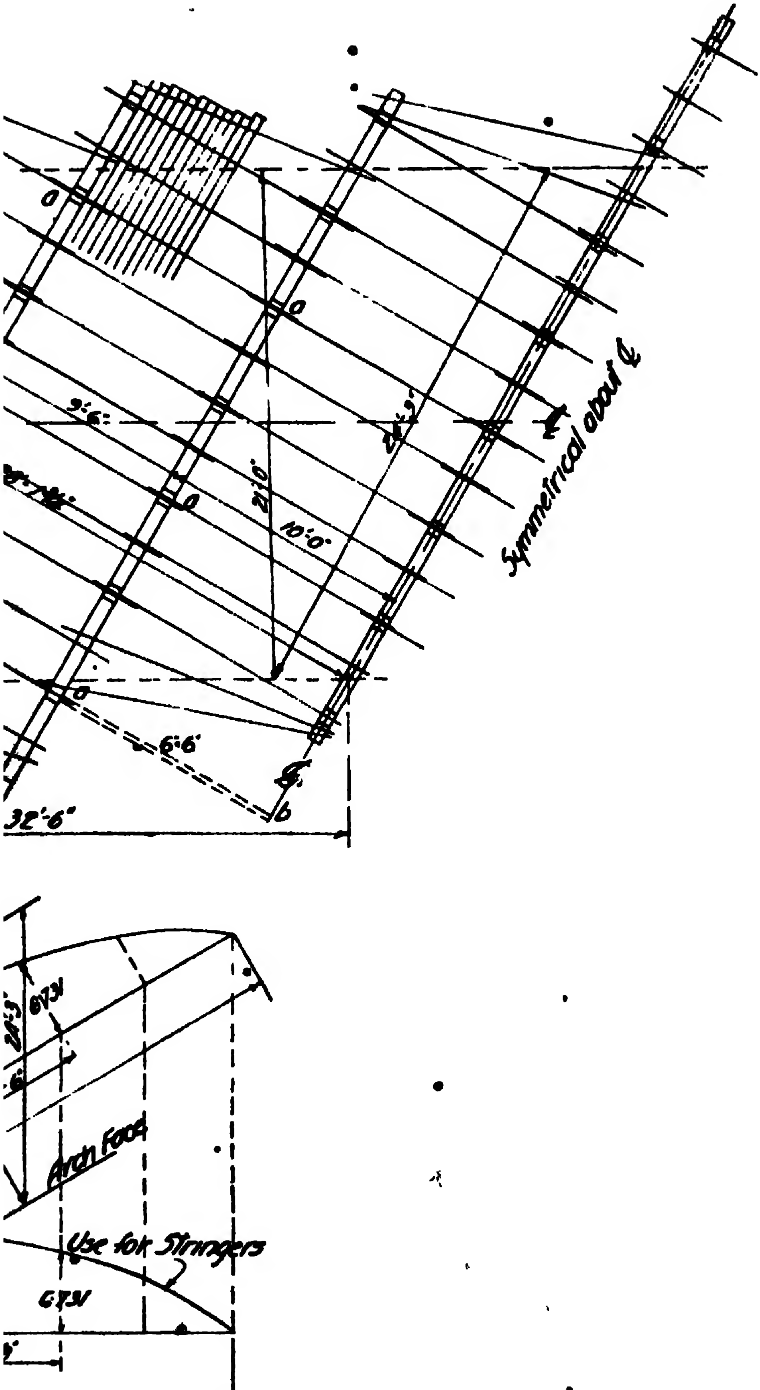
Coming now to solid-barrel arches, *Fig.* 159 shows a typical centering design used for a bridge of three 70 ft. spans, 40 ft. wide. One arch was built in the dry, one in water 9 ft. deep, and the other half in water and

half in the dry. *Fig.* 159 shows the latter condition ; the right-hand half is typical for the deep-water arch ; and the left-hand half for that built on high ground. The river bottom was rock, covered with 2 ft. to 3 ft. of sand and mud. The posts were driven by hand to rock, without using sills. One end of the braces was nailed to the posts before driving. Where the rock was clear the posts had timber sills with bags of dry concrete tied to them to hold them down. On dry ground individual sills were used, as the ground was too uneven for continuous timbers. They were built of 2-in. plank nailed to three 6 in. by 6 in. timbers 4 ft. long and 21 in. on centre. The post rested on a 6 in. by 6 in., 4 ft. long, laid at right-angles to the timbers. In other respects the design was similar to that shown in *Fig.* 154, and the same timbers were used. *Fig.* 160 shows a view of one of the centres.

FIG. 160.—CENTERING FOR A 70-FT. SPAN SOLID-BARREL ARCH, DESIGN FOR WHICH IS SHOWN IN FIG. 156.

Centres for skew arches, either ribbed or solid-barrel, can be designed in the same manner, only the stringers must be placed at right-angles to the abutment or pier and not parallel to the longitudinal axis of the arch, so that they will bear squarely on the caps. This will necessitate wider bents, overlapping the arch ring. *Fig.* 161 shows a design for a skew bridge of three 65-ft. spans with solid barrels. There was 3 ft. to 5 ft. of water, with good gravel bottom. Each post had an individual sill of cross-piece and bag of concrete. The 2 in. by 6 in. on top of the posts tied them together and formed a seat for the wedges. As the wedges prevent the caps being cleated to the posts, to prevent possible overturning of the caps on the bents near the abutments they were strutted against the adjoining bents at *aa*, and on the ends where no adjoining bents were available the thrust was transferred to the river bed by diagonal struts, as at *ab*. *Fig.* 162 shows a view of the centering during construction. As the stringers are at right-angles to the abutments they cannot be cut to the true curve of the arch.

FIG. 161.



Symmetrical about ℄
21'-0"
10'-0"
9'-6"
6'-6"
32'-6"
Arch Face
Use for Stringers
6.731

The method of developing the curve to which the stringers must be cut is shown in *Fig.* 161. The true curve is laid out by ordinates from a line making the same angle with the horizontal as the skew of the arch, or the angle between the longitudinal axis and the face of the abutment. If the points on the line at which the ordinates are taken are projected vertically downwards to the horizontal and the same value of the ordinates laid off, the curve joining the points obtained will be the "working curve" of the arch to which the stringers are cut.

Practical Details.

The first consideration in the field is laying out the centering so that the stringers can be cut accurately. This can be done on a level plat-

FIG. 162. CENTERING FOR A SKEW-BARREL ARCH OF 65 FT. SPAN.

form of 1 in. boards 4 ft. or 5 ft. wide, laid roughly to the shape of the arch soffit. The arch soffit is marked out on the platform to full size, only half the arch being necessary if symmetrical, by means of the radii or by vertical ordinates from the springing line. Some settlement will take place, so allowance must be made for it by giving the centre a camber, adjusting the true curve to this camber. The amount of camber depends on the span, height, and foundation conditions and method of pouring. About 1 in. to $1\frac{1}{2}$ in. per hundred feet of span is a suitable camber. Each joint in bearing will take up $\frac{1}{32}$ in. to $\frac{1}{16}$ in., due to tightening up of the centering when loaded and the biting of one timber into another.

On the adjusted curve the lagging, stringers, and caps should be laid out accurately. The required square bearing area of the stringer should be given on the centering plan; this will determine the amount to notch

out each end. Templates should be cut for each typical stringer. Instead of cutting the stringers out of single timbers, if the radius of curvature is less than 150 ft. curved strips may be nailed to square timbers, so that full salvage of the latter is obtained.

Saw cuts should be made true so that all parts will fit together tightly and squarely; this is particularly true of posts. The height of a post above the springing line can be obtained from the template and the depth below from levels taken at each foundation.

The posts are only temporarily braced at first, the final bracing not being completed until the stringers are wedged up to the correct elevation. Whether to erect posts singly or in bents depends on the means for handling them. The former is generally the cheapest, unless perhaps a cableway is used.

The cap and sills can be fastened to the posts by iron dowels or by nailing 2-in. cleats at each side; the latter is better and can be stripped more easily Stringers should be toe-nailed to the caps and wedged tight at the notches. Bridging is not necessary except with exceptionally deep stringers. *Fig.* 162 shows stringers in place on the caps.

Whether to use nails or bolts is a matter of individual preference. The labour cost of drilling holes, fitting and tightening bolts, washers, and nuts is high, while there is little salvage as the parts are easily lost, particularly over water. It is also difficult to tell whether the bolts are tight without testing each one. It is cheaper to use 20*d.* nails or spikes, and they will give a stiffer structure. Braces should be spiked with four spikes at each end, and kept high enough above the water to allow passage for débris and ice if necessary.

Wedges should be in pairs, of hardwood or long-leaf pine, tapering about 1 in 5. Eight inches wide, 12 in. long tapering from 3 in. to ½ in., is a satisfactory size, although it will depend on the size of posts. Wedges are sometimes made in three pieces instead of two. They should generally be placed at right-angles to the caps, not parallel, as they can be more easily adjusted that way. If the centering is more than one tier high wedges should be placed under the top tier of posts. If the posts are in one length they should be under the posts when possible, as they are much more easily handled than when they are at the top, although there will be more weight to lift; the posts and caps can also then be cleated together. Wedges should be inspected just before concreting, and should be watched during concreting, tightening when necessary.

Screw jacks and sand boxes are seldom used as a means of striking the centering. A row of posts can often be saved at the piers and abutments by bolting the caps to the concrete, the bolts being set in the forms.

Arch lagging is generally nailed on lightly in place, but may be built up in panels if it is to be used several times. It should extend about 2 ft. beyond the arch to form a gangway and footing for the side-form braces. In ribbed arches a board should extend under both ribs to form a tie at every 3 ft. or 4 ft.

The lower portion of the arch on each side may sometimes be formed

advantageously with the abutments or piers, especially when the curvature is steep at these points.

Lagging should be placed as soon as possible, so that the steel workers can follow on with the reinforcing bars.

Multiple Arches.

When there is more than one arch there is the question of how many full sets of centering should be provided. This is often specified by the engineer, as it will be governed somewhat by the arch design. With two spans it is nearly always necessary to build two complete centres. Occasionally, when the rise is high compared with the span, and the horizontal thrust on the pier consequently low, it may be possible to use only one centre, by providing a temporary reaction to the thrust. This was done with the bridge shown in *Fig.* 151. Cables were passed around the ribs from abutment to pier at the springing level, and a platform was built at the centre and loaded sufficiently to provide the necessary tension in the cables to enable one arch to be stripped and the timber used again in the second arch; the cables and weights can be seen in *Fig.* 153. In this case it could be done safely and more cheaply than providing sufficient timber for another centre.

With three spans, two centres are built, using the first centre for the third arch. With four arches, two or three centres are used. With five or more arches, three or more centres are used, depending mainly on the speed required.

For wide bridges it is sometimes economical to build the centres for half the width of the bridge and move them sideways for the other half, instead of moving a complete set of centres forward. This is done by using double caps on top of the posts, with the wedges placed between them. Before striking the wedges, short pieces of 1½ in. or 2 in. pipe are inserted between the caps to act as rollers. Striking the wedges will let the upper caps down on to the rollers so that the caps, stringers and lagging can be rolled over to the adjacent bents. In the first erection the bents should extend beyond the bulkhead far enough for the stringers to be rolled over sufficiently to release the outside posts and caps, which can then be erected ahead to complete the second half of the centering. When the bents are two tiers high the double caps may be under the upper posts, so that the whole of the upper half of the centering may be rolled over. This system is best adapted to short span wide roadway bridges.

In a 60 ft. wide bridge, the centering may be built for a width of 20 ft. and moved over twice. Care has to be taken to keep the centering moving evenly without distortion. After moving over the wedges are again inserted and the centering wedged up to line.

Stripping.

Wedges should be struck in pairs from the crown inwards to the springings, loosening them gradually without shock to the arch ring. About half the sway bracing can be removed in 7 days, and used in the

adjoining arch. Centres should not be struck under two to four weeks, three weeks being the usual time in summer for arches under 100 ft. span. Arches with high rise in proportion to the span can be stripped earlier than low arches.

All braces are removed before stripping the timbers, except those that

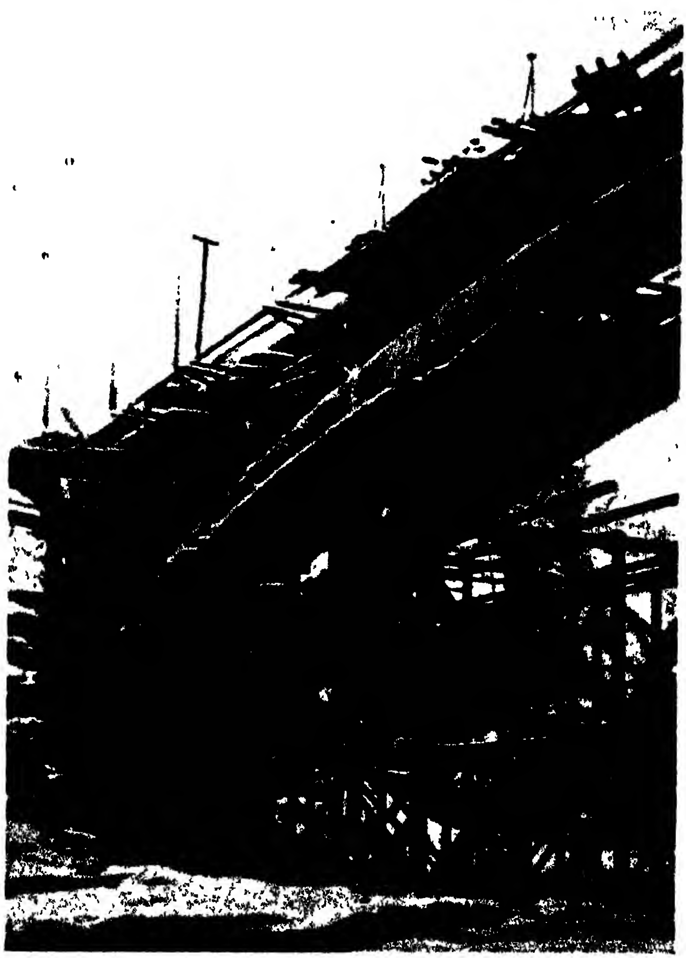


FIG. 163 METHOD OF STRIPPING HIGH ARCH

may be required for supporting scaffold plank. Low arches can be stripped bent by bent, pulling over a bent and knocking apart on the ground when possible. Over water the timbers will generally have to be taken apart in place.

With high arches the stripping should commence at the top, taking down tier by tier. An interesting method was used in stripping the arch shown in *Fig.* 155. Ropes were passed around the ribs under the ends

of the stringers, and tied. After the braces were knocked off the bents were pulled over bodily by ropes, tier by tier. The timbers were taken apart and dropped on to the ice, then the lagging and stringers were lowered down together (*Fig.* 163). Two carpenters and 5 labourers removed the centering and piled up the timber on the bank in 4 days.

With multiple arches a centering must not be struck and stripped before the adjoining arch is poured, and sometimes it is required that one or two arches be undisturbed between the arch last poured and the arch being stripped.

Bowstring Girders.

Overhead arches, or bowstring girders, where the roadway is suspended from the arch, consist of beam, girder, slab, and column units, and are built accordingly. The deck forms are built first, supported by bents or by individual shores, as in buildings, when the bridge is low enough. After the deck is poured, the vertical suspension members are formed similarly to columns, and the arch members like beams The sides of the beams must be tied across the top and will perhaps require back-forms when the slope is steep.

Fig. 164 shows the details of the forms used for an overhead arch of 120 ft. span and 26 ft. rise, and a 20 ft. roadway and two 6 ft. cantilever sidewalks. The arch centering was laid out on the ground and the forms built accurately in sections and numbered to save time in erection. In each bay three double 6 in by 6 in shores supported the arch rib, resting on wedges on plank sills. Ordinary 4 in. by 4 in. shores supported the deck beams from the rock bottom. The danger, however, of placing shores close together in winter time is shown in the photograph, where some of the shores are seen carried away by the ice.

Steel Centres.

When it is impossible to use bents and it is necessary to go into a trussed type of centering, steel trusses should be given consideration. When there are a large number of arches, even though bents could be used, steel trusses may be more economical owing to the saving in labour of erection. This is particularly true when there is deep water under the arches. With low arches and a river subject to high floods, steel trusses will offer less resistance to the flow than will any other type of centering.

The design of the trusses will depend on the design of the arches, the method used to handle them, and on local conditions. In general, they are built up in sections with light angles. In most cases tie members will be necessary to tie together the ends of the trusses, and these ties are suspended by rods from the bottom chord. to prevent sagging.

The top chord may be curved to the soffit of the arch, but is more usually in straight sections, as the trusses will then have more salvage value and can be more easily adjusted to suit different spans. The lagging may be supported in different ways. If the trusses are placed fairly close

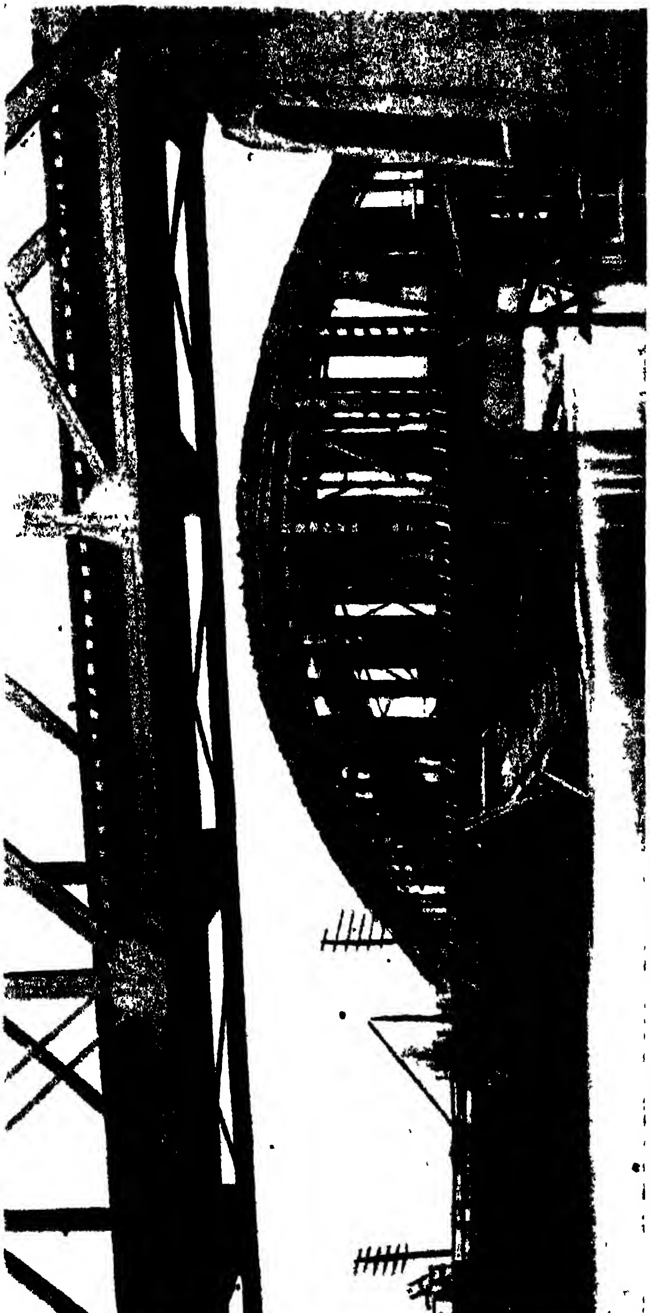
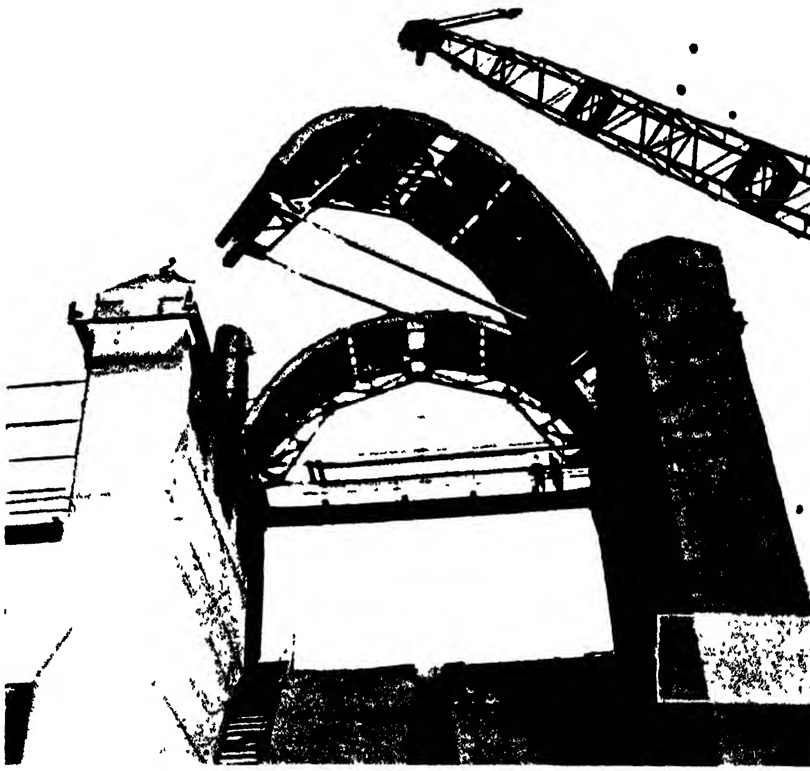


FIG. 164.—FORMS FOR 120-FT SPAN BOWSTRING ARCH BRIDGE

on centre, the lagging can span between trusses resting on curved wooden strips bolted to the top chords; in this case 3 in. or 4 in. lagging is used (*Fig.* 165).

For arches with high rise, the ordinary cap and curved stringer method may be used (*Fig.* 166).

For arches with comparatively low rise the lagging may be carried on



plank stringers bent over the caps to the curve of the arch, the caps being blocked up where necessary at the panel points (*Figs.* 167 and 168).

There are three main methods of handling the trusses: by a travelling derrick running on a trestle parallel to the bridge, which is the method most generally used (*Fig.* 168); by an overhead cableway, used for high bridges and when there is deep water (*Fig.* 167), and by means of stationary derricks set up for each arch on the ground or on top of the piers. Another method sometimes used in deep water is to float out the trusses on a raft, in which case they are mounted on a frame on which they can be raised or lowered (*Fig.* 166).

Method of Loading the Centering.

Some thought must be given to the method of pouring the arch so that the centering will not become distorted.

The best method is to pour the arch or ribs in alternate blocks running transversely for the whole width of the arch or ribs. Corresponding blocks on each side of the centre line must be poured together at the same

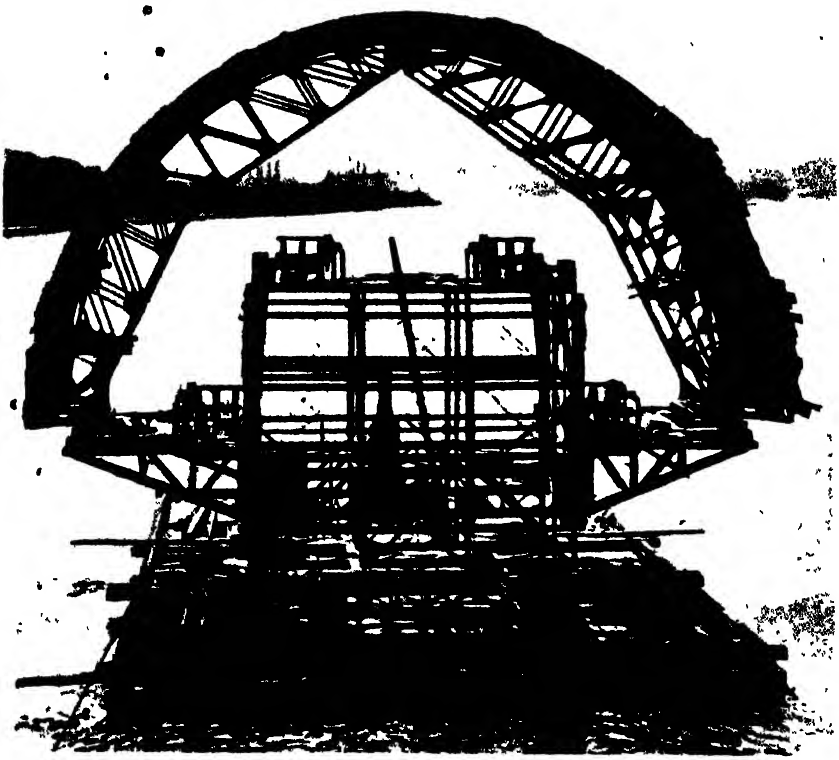


FIG. 160.—HANDLING STEEL TRUSS CENTRES BY FLOATING INTO POSITION WOOD CAPS, STRINGERS AND LAGGING.

rate, and in ribbed arches corresponding blocks in both ribs must be poured together in order to load the centering equally. The size of the blocks in large arches should be such that a set can be completed in one day. There will always be at least five blocks to an arch, two at the haunches, two at the springings, and one at the crown, the haunch blocks being poured first, then the springing blocks, and last the crown. This is in order to provide some weight near the crown so that the centre will not rise when the end blocks are poured, which is a condition that always has to be watched.

It is better still, especially in long span arches, to pour in seven sections,

FIG 167.—HANDLING STEEL TRUSS CENTRES BY CABLEWAY. PLANK STRINGERS BENT TO THE CURVE OVER TIMBER CAPS.

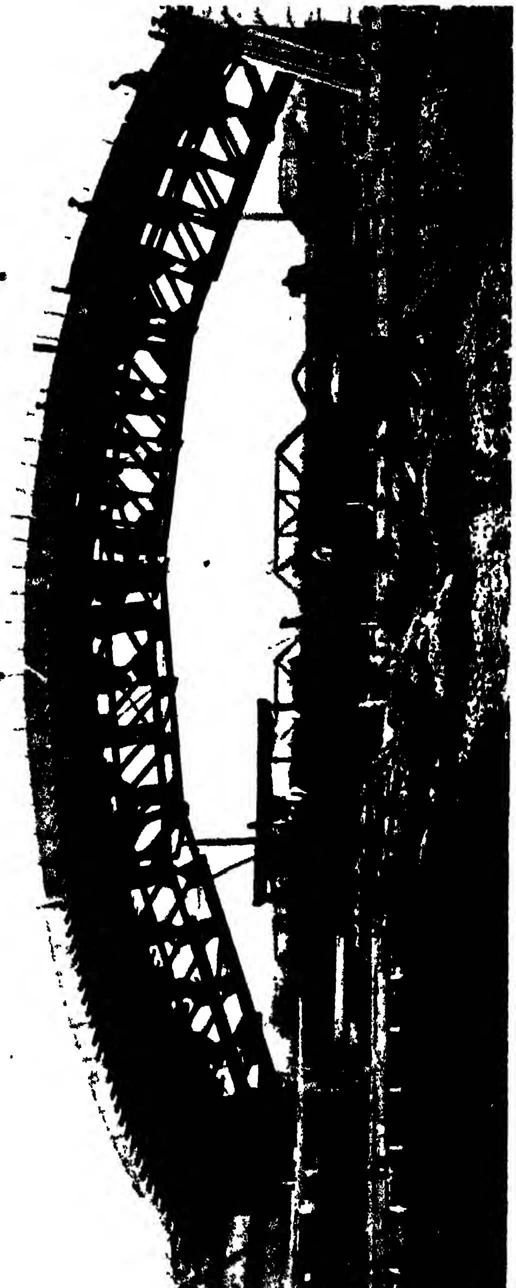


Fig 168.—Standard Steel Trusses adapted for different Arches, Plank Stringers on Timber Caps Erection by Travelling Crane on Low Trestle parallel with Bridge

in which case the sides of the crown sections are poured first, then the two nearest the springings, then the two between these sections, and last the crown section.

Pouring arches in narrow longitudinal ribs is not a particularly good method, as it is more expensive and more difficult to prevent distortion, and greater stiffness of the centering is necessary. In this method, and when a small arch is poured completely in a day, pouring must be done at the same rate from each abutment to the crown to load the centering symmetrically, and a temporary load may be required at the crown to prevent it rising.

With steel centres the arch or ribs should always be poured in transverse blocks to avoid cracks which may occur in the arch due to slight expansion and contraction of the steel under changes in temperature.

Estimating Cost of Centering.

Footings, foundation work, lagging and material for wedges should be estimated separately from the centering proper. Concrete work in footings, the driving of piles and the placing of timber sills can be estimated fairly closely, but whatever foundation method is used a certain lump-sum allowance should be added to cover what cannot be estimated closely.

Lagging is best estimated by the square foot laid for both material and labour, allowing for the overhang on each side of the arch or ribs. To place and strip 100 sq. ft of lagging will require 2 to 3 hours time of a carpenter and labourer for 1 in. and 2 in. lagging respectively, and proportionally for intermediate thicknesses.

The centering proper cannot be estimated closely without first making an approximate design and sketch and calculating the amount of timber required, as hardly any two bridges are alike in all respects. A rough estimate of the amount of timber required can be made by allowing ½ cu. ft. per sq. ft. of area supported by the centering ; that is, the overall width of the bridge multiplied by the clear span. As a guide, the amount of timber used in *Figs.* 151 to 161 is given, with and without piles, but exclusive of sills, foundations, wedges, and lagging.

Design in

Fig. 151 required (without piles) 970 cu. ft. = ·40 c f. per sq. ft.
Fig. 151 „ (with piles) 1250 cu. ft. = ·51 c.f. per sq. ft.
Fig. 154 „ 4200 cu. ft. = 0·62 cu. ft. per sq. ft.
Fig. 157 „ 4000 cu. ft. = 0·59 „ „ „
Fig. 158 „ 1600 cu. ft. = 0·51 „ „ „
Fig. 159 „ (for arch over dry ground) 1075 cu. ft. = 0·39 cu. ft. per sq. ft.
Fig. 159 „ (for arch over water) 1300 cu. ft. = ·46 cu. ft. per sq. ft.
Fig. 161 „ 650 cu. ft. = ·47 cu. ft. per sq. ft.

The labour cost is calculated on the amount of timber estimated.

When vertical posts are used the cost is not much affected by the span and rise.

To erect and strip 100 cu. ft. of timber will require about 27 hours time of a carpenter and labourer. When short inclined posts are used in the top tier of bents, about 10 per cent. should be added to this time. When long inclined posts are used about 20 to 25 per cent. should be added.

Oiling of lagging should be estimated separately.

CHAPTER XIX.

OTHER BRIDGE FORMS.

APART from arch centering, bridge forms are in the main similar to corresponding forms in building construction, so that only the details required to adapt these forms to bridges need be considered.

More attention has to be paid to obtaining straight lines and true surfaces, since any irregularity is very noticeable in a bridge. When setting the forms it is best to work to as long a line as possible, as short lines tend to produce kinks in the concrete. The lines, especially of copings, should be watched all the time concrete is being poured so that slight adjustments can be made if necessary before the concrete sets. New matched timber should be used for all exposed faces, and if used more than once should be well cleaned each time. Exposed faces should be stripped when possible within 48 hours so that the concrete can be rubbed down while still green

Abutments and piers have been mentioned in the chapters on walls and dams, and these chapters will also explain how to build forms for wing walls.

Arch centering is often stripped for use on another arch before the spandrel walls or columns and deck are built, in which case these forms have to be supported from the concrete alone.

Before pouring the arch ring, consideration should be given to the design of the spandrel forms, so that bolts or wires can be left in the concrete where required to serve as anchors for foot-blocks and shores, otherwise much extra bracing may be required.

Pier Nosings.--Pier nosings or cutwaters are generally built up into a unit. A curved form is the most usual shape. The horizontal yokes, generally of 1 in. material, are cut to template, the sections overlapping and being nailed together. The sheathing generally cannot be more than 2 in. wide, and the boards must be tapered towards the top if the pier batters.

The yokes should project beyond the sheathing so that they can be nailed to the side wales (*Fig.* 169). The lower yokes should be wired to iron rods set in the footing, and in addition all yokes may be wired across to the side wales. Vertical timbers placed against the yokes will stiffen the form.

Nosing caps are generally half a pyramid or cone in shape. They should be formed and poured with the piers, not with the spandrel walls, often being part of the skew-back forms for the arch (*Fig.* 170). They

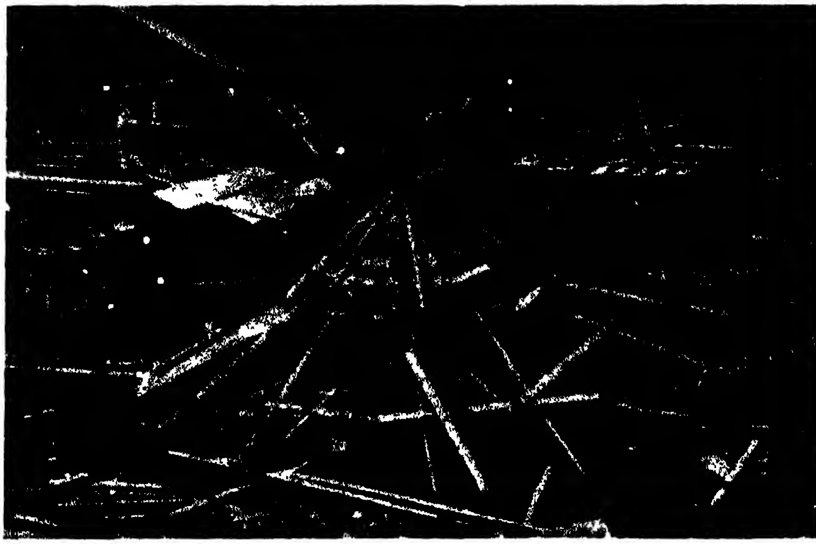


FIG. 169.—CURVED PIER NOSING FORM.

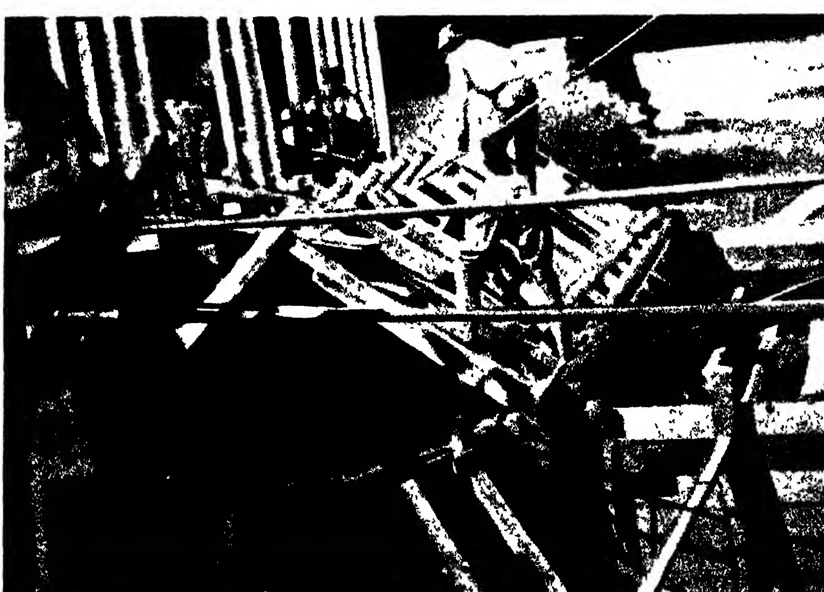


FIG. 170.—ARCH SKEW BACK FORMS TOP OF PIER.

can often be advantageously formed with sheet metal; if not, 1 in. ribs are used with narrow sheathing coming to a point at one end. Flat caps can be built up to a template without a form.

Arch Side Forms.—Arch side sheathing should generally run horizontally, and not vertically. The form can be made up in short panels with studs attached, using the centering template for cutting the curve; or the studs can be erected first and temporarily braced, afterwards cutting and nailing on the boards. The panel method will give the best job provided the arch is built true to the curve. The studs are usually placed vertically except for a few near the springings; but are sometimes placed radially, which avoids the necessity of cutting the bottoms of the studs to fit the curve. The joint of the side form with the arch lagging is always covered with a bevel strip. If the curve is sharp, the bevel strip may be bent without breaking by first soaking well in water.

The lower ends of the studs, cut to fit the curve, are held by 1 in. by 6 in. ribbons nailed to the lagging Wales must be used to keep the form in line. In barrel-arches they can be wired to the bottom arch steel, and in addition be braced from the lagging. In rib arches they are wired or bolted together across the ribs.

Side forms can be seen in several of the figures of the previous chapter.

Arch Backforms.—Backforms will be necessary when the arch is steep. They can be built lightly, as there will be little pressure on them. With ribbed arches, short boards are nailed on top of the upper bevel strips as the concrete is poured. If the ribs are so wide that the top boards may bend, a centre 1-in. batten can be used, wiring it to the top steel of the arch. With barrel arches the backform is made up in panels, 2 ft. to 3 ft. wide and as long as convenient, setting them as required (*see Fig.* 73, Chapter XI).

Arch Bulkheads.—Bulkhead forms will be necessary at construction joints, either longitudinal or transverse. If the arch is poured in longitudinal rings the bulkhead form will be similar to that for the sides. If built across the arch it will be in three parts. A straight strip is placed under the bottom reinforcing steel and another one on top of the top steel. Between these strips, panels are set with notches cut out for the reinforcing rods. Battens hold the three parts together and a few diagonal braces will hold the form in place.

Spandrel Wall Forms.—The sheathing is run horizontally and is made up into panels between expansion joints. The wall will either be flush with the face of the arch or, more usually, set back an inch or two. In the former case the sheathing will overlap the arch and need not be sawn to the curve. When the wall sets back the sheathing must be shaped to the arch ring. In either case the studs should extend below the top of the arch so that they can bear against the face, notching them out at the bottom when necessary. They are held by a wale bolted to the arch concrete (*Figs.* 171 and 172). This wale can be in short straight lengths or consist of two planks which can be bent to the curve, with the bolts

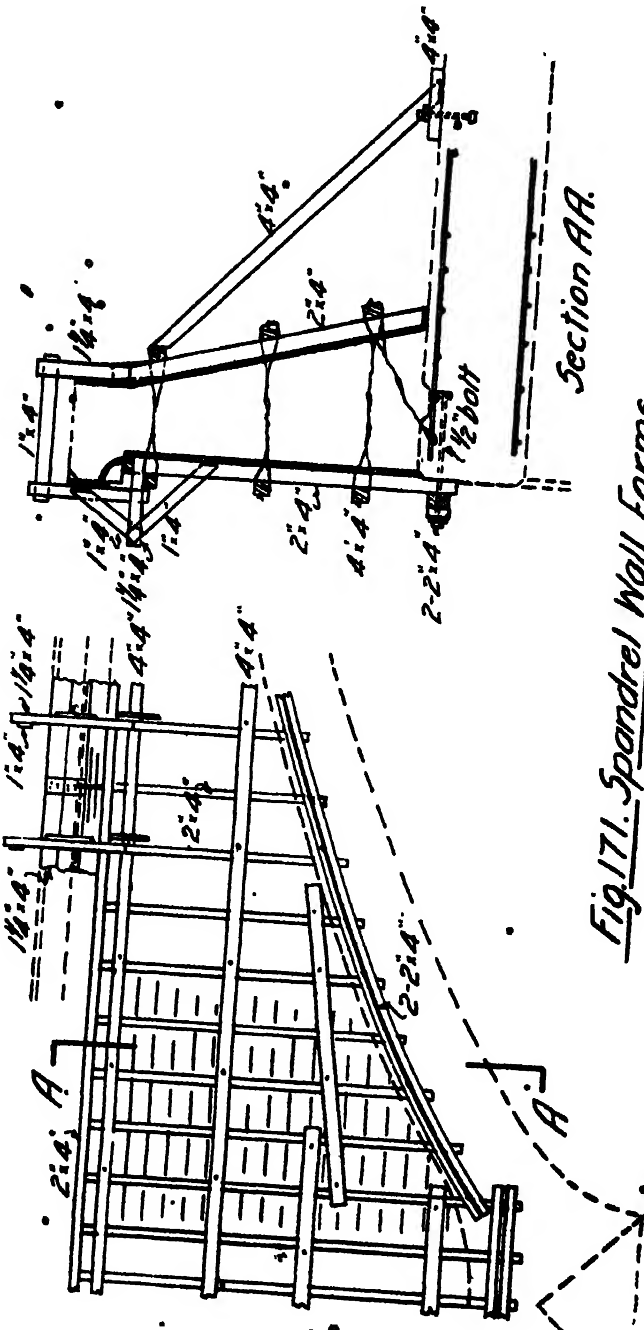


Fig. 171. Spandrel Wall Forms.

Fig. 172.—Outside Spandrel Wall Form.

between them. The other wales can run horizontal or partly on the skew. The whole outside form should be set first and lined up, so that it can be set to one long line. The inside form need only be set up as required between expansion joints. The studs should be nailed to a curved board laid along the top of the arch ; this will prevent concrete escaping under the sheathing. If there is considerable batter on the form it may have a tendency to uplift, and this can be prevented by wiring the lowest wale down to the concrete by wires left in between the forms. The inside form should be shored back to the arch ring, the ends of the shores being nailed to footblocks, wired or bolted to the concrete (*Fig.* 173).

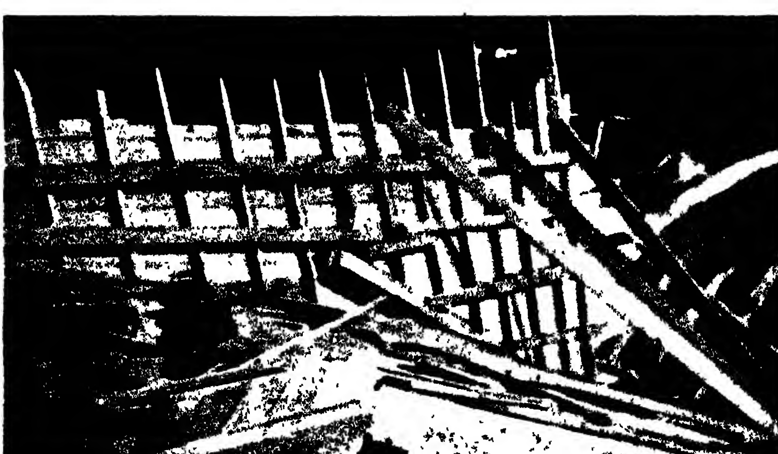


Fig. 173.—Inside Spandrel Wall Form.

The coping is formed by notching out the studs if the overhang is small. If too large for this method, the coping form is supported on brackets nailed to the studs (*Fig.* 171). Coping forms must be well braced and lined up carefully. To obtain good lines on the face it is best to use sheathing at least 1¼ in. thick, as it will be less flexible than 1-in. boards.

Open Spandrel Deck Forms.—If the arch is a solid barrel supporting a beam and girder roadway on walls or columns, the forms will have a solid support and are built in the same manner as in building construction. The posts should be set on longitudinal boards or nailing strips, and should be well braced and blocked to prevent slipping on the curved surface. Wedges should always be struck under the centering before building the forms for the deck, otherwise striking the wedges and consequent settling of the arch will throw the forms out of line.

The deck forms for ribbed arches present more difficulty, as they have to be supported from narrow ribs or from the centering. The latter method is not often used, because it is not usually economical to carry the deck supports right to the foundation, and it also means tying up the centering until the deck forms can be stripped.

However, this method may be suitable in the case of a one- or two-span bridge of low rise, where the centering is only used once. Additional posts will be required in the bents between ribs to carry the shores for the beams.

More often it is desirable to strip the centering as early as possible, both to use on another arch and to prevent possible damage from floods, before the deck forms are commenced, in which case the latter can be supported from the ribs only. The design then will depend on the design of the superstructure, the width of the ribs, and the distance between them. It will always be possible to shore some of the beams directly from the ribs; the main difficulty will be with the beams that come between ribs and with cantilever sidewalks. Timbers can be placed from rib to rib to support the shores, but generally they have to be so heavy to carry the concentrated loads without deflection that the method is not economical. Also, it is difficult to set these timbers level on a sloping surface, as they have to be blocked up and well braced.

A good construction is shown in *Fig.* 174, taking the deck over the arch shown in *Fig.* 154 as an example. Vertical posts support the beams where possible. The centre beam is supported by "A" frames, made of two inclined posts and braces. All posts are in line so that they can be securely braced together in bents. Wedges for adjusting the beam boxes are placed between two caps, the upper wedge serving merely as a bearing timber and the lower carrying the load. When the posts cannot be placed directly under the beams, the lower cap must, of course, be heavy enough to carry the concentrated loads. The two caps can be cleated together after the wedges have been finally adjusted.

To reduce the number of post bents required, the beam bottoms, instead of being 2-in. planks as would ordinarily be used, consist of 1-in.

boards nailed to two timbers on edge, thus allowing a longer span between shores. Beam sides should extend to the bottom of these timbers, and, if made sufficiently stiff, they will carry the load from the slab.

Posts should rest on plank or board sills and be toe-nailed to them, with blocks or struts to prevent them slipping down the slope.

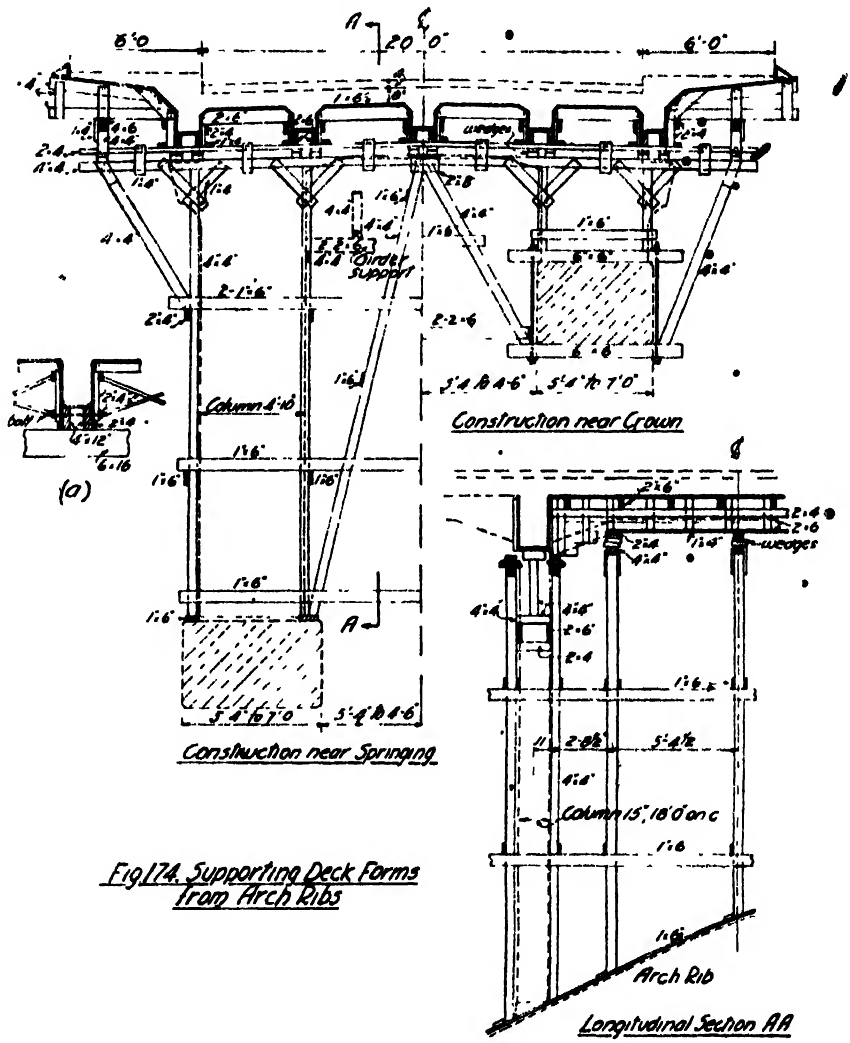


Fig. 174. Supporting Deck Forms from Arch Ribs

For some distance each side the crown the depth from the top of the ribs to the bottom of the beams will often be too small to support the "A" frames on top of the ribs, as the posts would be at too flat an angle. A steeper angle can be obtained by bolting yokes over the ribs as shown and supporting the "A" frames from the bottom members, securely tying the bottom of the posts together.

The cantilever sidewalks are supported from the same caps by extending them out and shoring the ends diagonally back to blocks on the outside posts. The angle of the shore with the horizontal should be 60 degrees or over. In addition to support from the blocks, the diagonal shores should be nailed or bolted to two cross braces extending across the bent. Near the crown these shores can be carried on yokes, as shown.

FIG. 175.—SUPPORTING DECK FORMS BY "A" FRAMES AND YOKES AROUND RIBS.

The sidewalk form is made up in panels on frames of 1 in. or 1½ in. boards and rests on a ledger on the outside beam form and on a timber spanning between shores. To give a steeper angle to the shores the sidewalk form is allowed to cantilever over a short distance, and to prevent any possible overturning the frame is nailed to the beam side battens, which are placed on edge.

FIG. 176.—SUPPORTING CANTILEVER SIDEWALK FORMS NEAR THE CROWN.

FIG. 177.—SUPPORTING CANTILEVER SIDEWALK FORMS NEAR THE ABUTMENT.

Figs. 175, 176, and 177 show the methods actually used on this bridge, which were necessarily somewhat different as use was made of the steel I-beams in the old bridge. An " A " frame at each column supported a 20 in. I-beam under the centre beam, running longitudinally, and in place of the 4 in. by 4 in. caps, 7 in. I-beams were used, supported over the ribs by the top chords of the old steel arch, around which the new bridge was built. Yokes around the ribs were used, as shown, near the crown.

When the ribs are very narrow and far apart and the arch is very high, a condition sometimes occurring in arch viaducts, the method described above would not be suitable as there would probably be three beams to support between ribs and the posts would be very long. In this case by using heavier timbers the posts may be omitted entirely. The beam bottom can be built in the same manner, but the stringers must be heavy enough to span from column to column, usually 16 ft. to 18 ft. At the columns they can be carried on heavy ledgers bolted to the concrete, one on each side, or supported by posts from the ribs. If posts are used they should be bolted to the columns.

In *Fig.* 174, if this method were used, two 4 in. by 12 in.'s would be required to carry the beams, resting on 6 in. by 16 in. ledgers, either bolted to the concrete or shored by posts at the centre of the ribs. This method can be used economically when the centering is designed with this in view, so that the timbers can be used again in the deck forms.

The beam sides can be bolted or nailed to the stringers, and should be cross-braced in each bay to stiffen the forms. A 2 in by 4 in. nailed to the side of the stringer will support the beam side during erection, or sides and bottom can be erected as a unit. This construction is shown at *a*, *Fig.* 178.

When there are brackets on the ends of the beams the stringers can be dropped sufficiently below them and the beam bottoms can be 2-in. plank running longitudinally, being blocked up from the stringers about every 3 ft.

Cantilever sidewalks sometimes occur with earth-filled arches when counterfort spandrel walls are used. The forms will be easier to construct as they can be carried on brackets nailed to the wall form studs.

When the walls are stripped before building the sidewalk forms, bolts should be left in the concrete for ledgers, one near the top of the wall and the other about 6 ft. lower, to which the brackets can be fastened. To give a steeper angle to the diagonal arm of the bracket it can be supported on a short timber bolted to the underside of the arch. *Fig.* 178 shows a method of building the forms for a sidewalk carried by cantilever beams at each counterfort. A bracket on each side of these beams will support the beam forms. Although it is easier to build the sidewalk and wall forms together, it means delaying the stripping of the walls until the sidewalk can be stripped.

Railing Forms.—The most noticed part of a bridge is the railing, and therefore attention must be paid to the forms on which its success will mainly depend. New timber should be used, and sheathing boards

should be 1½ in. to 2 in. thick, as they are easier to set true to line than thinner boards and require less repair with repeated usage. A timber easy to work and with little grain should be chosen, white pine or spruce being very suitable.

Long lines should be worked to, and measurements should not be made from the faces of the wall coping, since any irregularities in it will

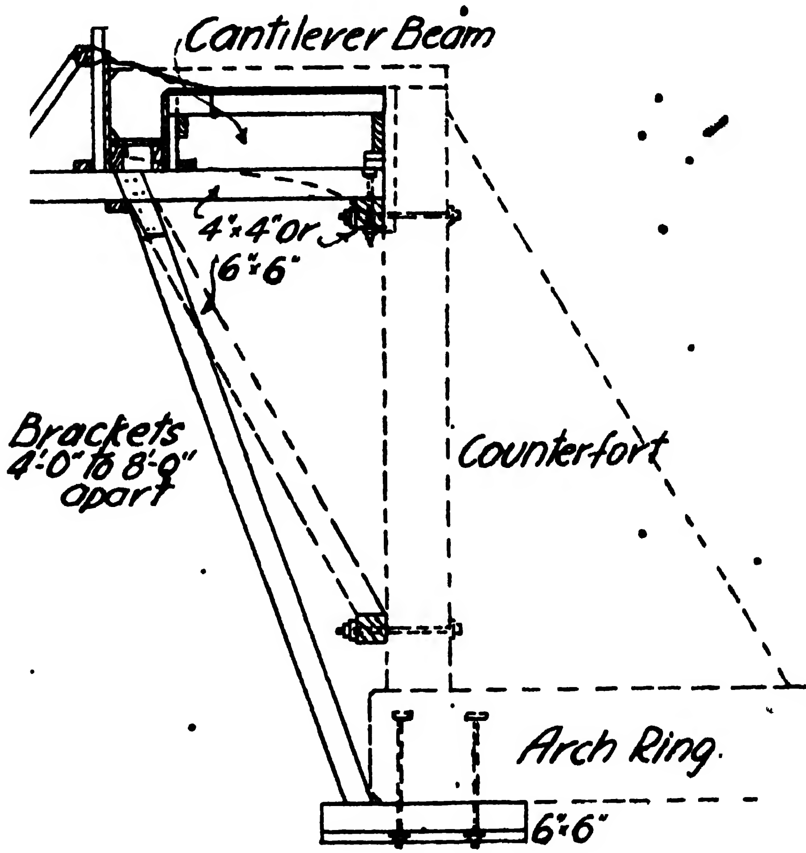


Fig. 178. Cantilever Sidewalk Forms

be repeated in the rail, where they are still more noticeable. Forms should not be set and poured in short sections where there are long unbroken lines, as every construction joint tends to produce a kink in the line. When there are intermediate posts breaking up the lines, the forms should be completed between posts before pouring.

Pouring in as long a section as possible is particularly desirable for base or plinth, and the coping along which the eye naturally travels.

The simplest type of rail is the plain solid wall merely relieved by inset panels. Various methods of building and supporting the forms are shown in *Fig.* 179. This type of wall may be set on a base, in which case the base form is set and poured first. A simple form of clamp will hold the sides (*Fig*. 180). As the form will only be a few inches high there will be little tendency to side movement, but a few wires embedded in the concrete and twisted over the clamp are useful.

The more elaborate rail consists of a plinth or base, a dado or central portion, and a top coping or rail. The dado is usually divided into panels between posts, and may be solid or pierced with openings, or consist of separately moulded balusters. The method of building the forms depends largely on the design of the rail. Except with a very simple design it is best to form and pour each part separately, as making one form to include base, dado, and coping is usually expensive and difficult to set and strip without breaking off corners.

When the base runs in an unbroken line or with only offsets at the posts it is set and poured first.

If the posts set on the wall coping and not on the base, they are formed first and the base built between them.

The form for a plain rectangular post is similar to a simple column form with the yokes nailed or clamped together (*Fig*. 181). They can be braced to stakes in the fill or to blocks bolted to the roadway slab, and can, if desired, be set in a template as shown to hold them square. Two methods of building a form for a post with base, dado, and cap are shown at *a* and *b* (*Fig*. 182). In the first case the whole post consists of one form, built with two sides and two ends and held together by yokes like a column form. In the second case the form is in four separate parts laid on top of each other. It will be found much easier to build than that shown at *a*, and has the advantage that each part can be stripped separately. Form *b* can be stripped before forms *a* and *c*, and this prevents breaking off the lower edges of the cap, which will tend to split off diagonally unless great care is taken.

A simple dado form for a plain wall with inset panels between posts is shown in *Fig*. 183. The base and posts are formed and poured first. (Note that if the coping were formed with the wall it would be difficult to strip the form without breaking off the lower edge of the coping.)

Dados pierced with openings should have the opening forms made as separate units apart from the side forms. They are then lightly nailed to one side form and the other side form butts against them (*Fig*. 184, *a*). To facilitate stripping, the opening form can be made in two halves as shown, with a bevelled edge board opposite the joints which can be removed first. The form will remain in the concrete when the sides are stripped and so protect the edges.

Sometimes these forms are split down the centre and each half nailed to a side form, but it is difficult to match up the sides exactly and a ridge will tend to form in the concrete at the joint, and when stripping the drag

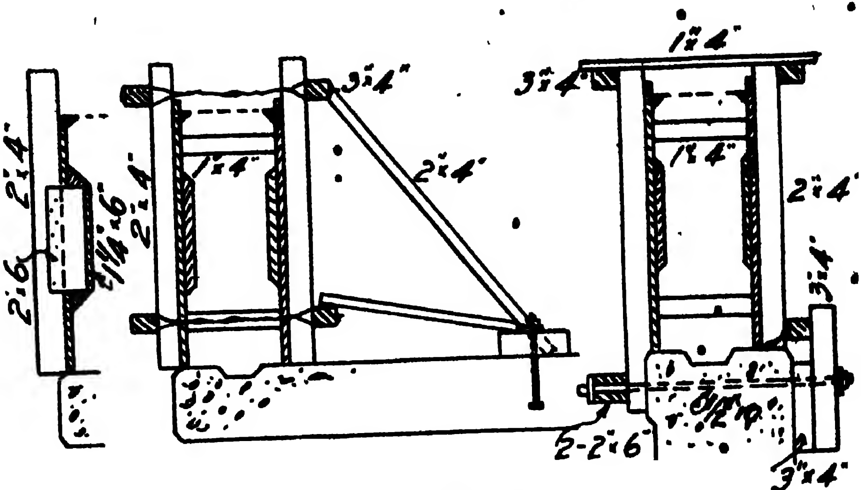


Fig. 179. Plain Rail Forms

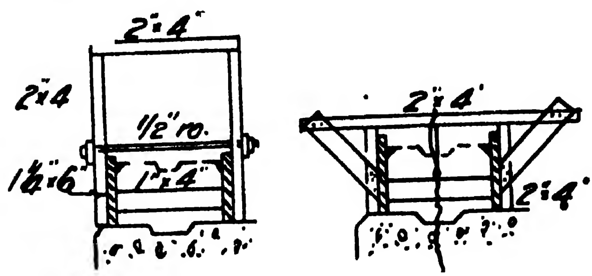


Fig. 180. Base Forms

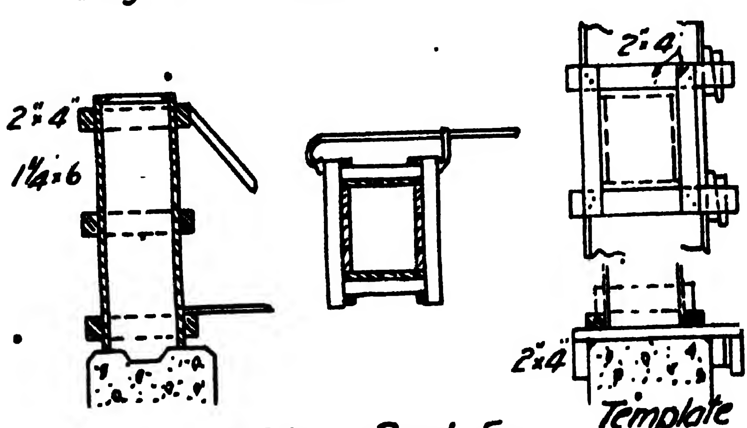


Fig. 181. Plain Post. Form

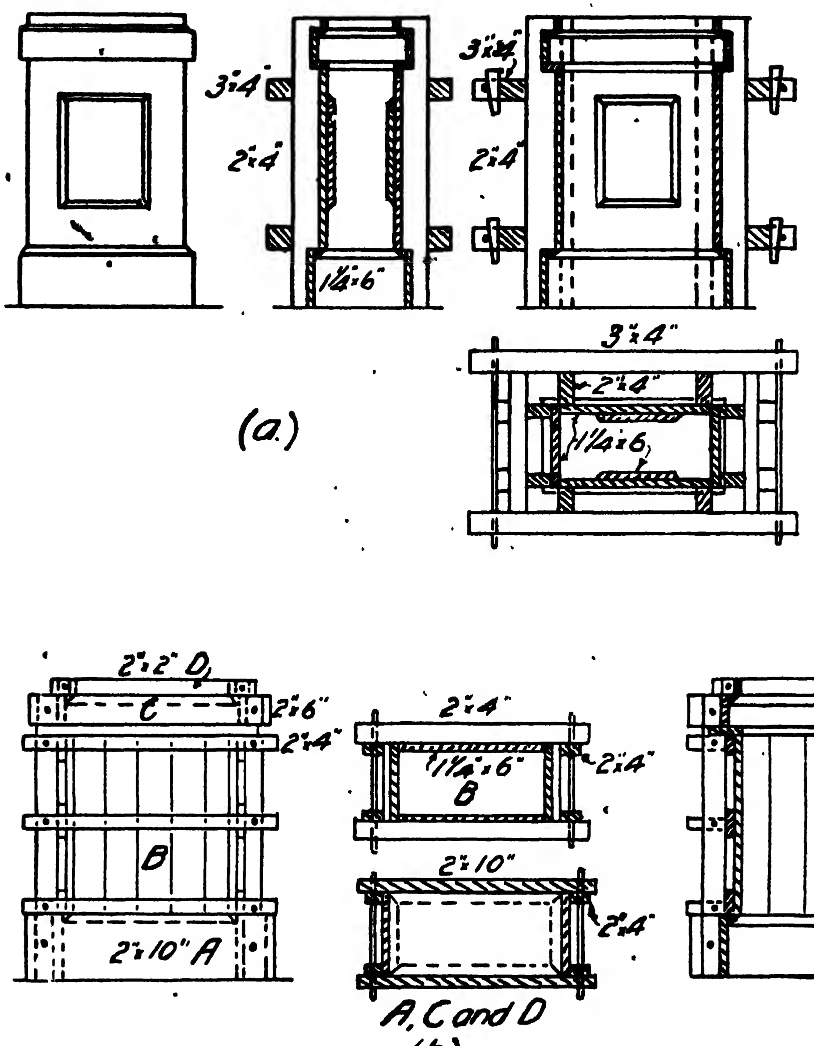


Fig. 182. Post Forms

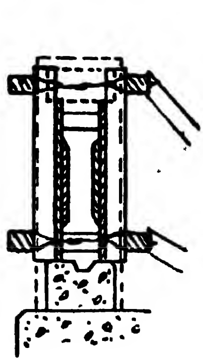


Fig 183. Solid Dado Form

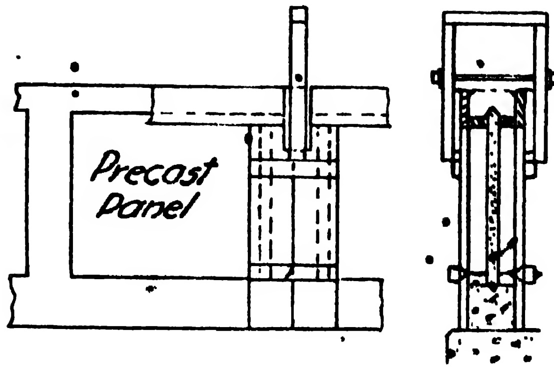


Fig 185. Coping and Post Form

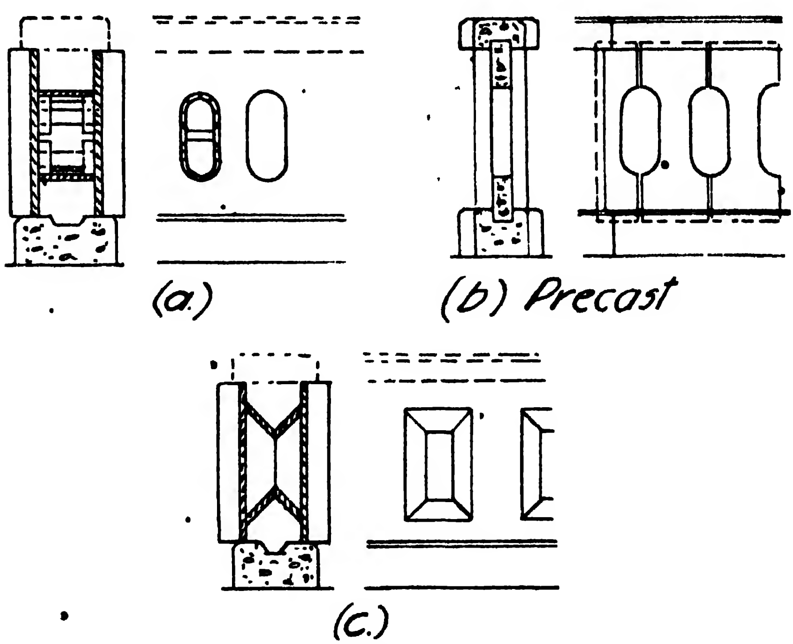


Fig: 184. Pierced Dado Form:

of the form may break off the edges of the concrete. This method can only be used successfully when the sides of the openings are bevelled to a point at the centre (*Fig.* 184, *c*).

A dado which looks like a solid wall pierced with openings may be split up into pre-cast units, with joints at the centre of the openings as shown at *b*, *Fig.* 184.

Dado forms are expensive to make up, and so they are used several times, but the coping form should not be set until there is a long length of dado ready so that a long unbroken line can be obtained. It is becoming more usual to build the dado 2 in. to 4 in. thick and pre-casting the panels between posts or in sections 3 ft. to 4 ft. long.

FIG. 186 PRE-CAST SOLID PANELS POSTS AND COPING FORMED TOGETHER.

A wall less than 4 in. thick cannot be successfully cast in place. This method can be used for solid panels or pierced panels. In this case the panels are set up on the base in a slot to receive them and temporarily braced, and the coping form is built around them. If there is a post the width of the coping between each panel, both posts and coping can be formed together (*Figs.* 185 and 186).

When the dado consists of moulded balusters these have to be pre-cast. Balusters are set in a bed of mortar, either a slot being left in the base to receive them or merely a hole for the centre reinforcing rod. Balusters should be set vertical whether the base is level or on a grade. After being set the coping is formed around them, using clamps to hold the sides. If the balusters are divided into panels by plain posts, these can be formed

with the coping (*Fig.* 187). Expansion joint material is set in the forms so that the forms need not necessarily end at the joints.

The top of the coping is best shaped with a wooden trowel made to fit the outline and not with a form, as better lines will be obtained.

Forms for Pre-cast Units.—Unless the shapes are simple in outline, pre-cast units are best left to specialists in this work. Knowledge not only of how to make the moulds but also of how to mix and pour the concrete is necessary, since often with the best built mould it is difficult to obtain a good surface, the chief trouble being with air bubbles and voids.

Wood moulds are used for flat panels and simple shapes. They are best made in a shop, where better workmanship can generally be obtained. The main point in their design is to allow for easy removal of the unit without injury and while still green. For moulded balusters, sand, cast-

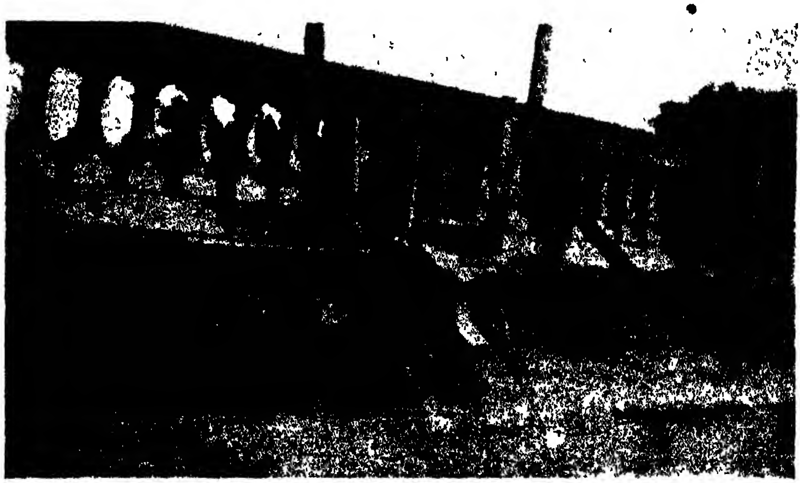


Fig. 187.—Post and Coping Forms with Pre-cast Balusters.

iron, or plaster moulds may be used. This is work for a specialist and not for a general contractor. Sand moulds will give good results but are expensive. Cast-iron moulds are good, and after a set of them is made the casting can be done quite easily on the job.

Cast-iron moulds should be about ⅜ in. thick and in two parts, the mould being split longitudinally and the two halves bolted together, with the faces in contact machined. At any place where it is difficult for the air to escape, as, for instance, at the top of the base, a small hole should be drilled in the casting. The form should be stripped within 48 hours, and the balusters can be finished successfully by rubbing with pieces of burlap soaked in water to which a little cement is added. *Fig.* 187 shows balusters cast in iron moulds on the job and finished in this way.

Concrete lamp-posts are cast in wooden forms. They can be pre-cast and then erected, but the erection will generally be more difficult than

erecting the form and casting in place, as the form is more easily braced and handled and a better connection can be made with the post or pedestal. The lamp-posts shown in *Fig.* 186 were first pre-cast but afterwards cast in place.

Ornaments.—Ornamental work cast in place should be confined to copings, cornices, belt courses, brackets, and simple panels. These forms have been described in previous chapters. Elaborate relief ornaments should be pre-cast and set in the forms or in recesses left for them.

Girder and Trestle Bridge Forms.—These require no particular mention, since they are similar to forms for building construction, using bent supports when necessary as in arch centering.

Estimating Cost.

Bridge forms are built under so many varying conditions that the same uniformity of cost cannot be expected as in building construction.

The amount of timber required per square foot will be approximately the same as given for the corresponding building forms. The number of times the timber or panels can be used will depend on the location and should be estimated for each operation. Sufficient timber should be provided and sufficient panels made up so that the operatives will not be waiting for panels to be stripped from another part of the structure.

Factors entering into the labour cost are height and length of the structure, accessibility, water conditions, method of handling forms by hand or machinery, and the number of times the forms can be used.

The highest unit cost will in general be for small one-span bridges, because the timber can be only used once or twice. With multiple arches the forms can be used several times and as the making up of the panels has to be done once only the unit cost is reduced. Also, with frequent repetition of the same operations, the carpenters become more efficient.

A high structure generally means lost time in climbing around and getting to the work and more difficult shoring and bracing.

The presence of water will always mean higher costs, particularly for the substructure, as the work will be less accessible, timber and panels will have to be carried a greater distance, workmen will move about more slowly, tools will be lost, and stripping will be done under greater difficulties. The use of cableways, travelling derricks, or other plant will reduce the cost of handling the forms, especially in large bridges.

It is only possible to give a range of unit labour costs, the actual cost to use for estimating depending on the judgment of the estimator of the difficulties of the work.

The number of labourers or carpenters' helpers will be approximately the same as the number of carpenters used on any particular operation, although certain work will be done entirely by carpenters and other work entirely by labourers. The number of hours required to build and strip the different kinds of forms is therefore given as the time for one carpenter and one labourer working together.

To build panels, erect, and strip 100 sq. ft. surface contact of forms will require approximately the following number of hours time of one carpenter and one labourer:

	Hours.
Piers, abutments, and wing walls	7 to 10
Pier nosings or cutwaters	14 ,, 18
Arch sides	10 ,, 13
,, backforms	4 ,, 6
,, bulkheads	8 ,, 10
,, spandrel walls	9 ,, 13
,, ,, ,, copings	14 ,, 20
Slabs	5 ,, 7
Beams and girders	11 ,, 14
Columns	10 ,, 13
Brackets (each) (carpenters only) . . .	7 ,, 14
Railing—base	9 ,, 12
posts	10 ,, 14
plain dado	9 ,, 12
balusters formed in place . . .	12 ,, 15
coping	12 ,, 15

CHAPTER XX.

PATENT DEVICES.

THERE are some operations in formwork construction that are slow and comparatively expensive, and which form a large proportion of the total cost of the work. For instance, wiring up a wall-form requires at least two men, possibly three, to handle each wire and spreader, and the cost of doing this work is quite a large proportion of the cost of erection. Making and placing shores and wedges is another item that is comparatively expensive, requiring three men for the erection of each shore.

To eliminate much of this labour, many patented devices have been placed on the market. Although the main idea of these devices is to save labour, in some cases the cost of material is also reduced.

Whether to use these patented articles or not mainly depends on the volume of concrete work the contractor is doing. In most cases the first outlay will cost more than using ordinary methods, so that their economy depends on the number of times they can be used. They should therefore in most cases be regarded as plant and be charged to the job in the same way. As plant they are a good investment. Practically all these devices are either of the nature of wall ties, column and timber clamps, or adjustable shores, and are practically all manufactured in America; they are not nearly so well known in this country as they deserve to be.

Wall Ties and Clamps.—The use of ordinary wire for tying wall forms necessitates the cutting of the wires to length, drilling four holes in the forms, passing the wires through one side and pulling through the other, twisting around the wale, twisting on the inside against the spreaders, cutting and placing spreaders, cutting off the ends of the wires, and patching the holes—quite a large number of operations.

If the walls are under water pressure the wires tend to form a passage for the water and rust spots appear on the face of the wall. Bolts are easier to place, but the first cost is high and they must either be left in the wall (when the ends are difficult to cut off) or they must be pulled; which, unless done as early as possible, may cost more than the material is worth and holes are left in the wall; and however carefully patched these holes may cause leaks.

With very narrow walls the spacing of the spreaders and the tightening of the wires within the forms is difficult or impossible, as there is little room to work, especially when the walls are reinforced.

The following clamps and ties are designed to remove some of these objections to wires and bolts.

Tyscrus (*Fig.* 188).—Manufactured by the Richmond Screw Anchor Co., of Brooklyn, N.Y.

"Tyscrus" consist of two No. 4 wires electrically welded each end to a helix of wire which forms a nut for ordinary ½ in. diameter lag screws. Their working strength is 5,000 lbs. They are made in lengths of 4 in. to 30 in., varying by 2 in. The wires act as spreaders as well as ties. For watertight walls, or where an exceptionally good finish is required, the ends of the wires are kept back 1 in. from the face. In either case the wire remains in the wall and the lag screws are removed. For panel construction with vertical lifts they form useful anchors, and they can be used for attaching anything desired to the wall.

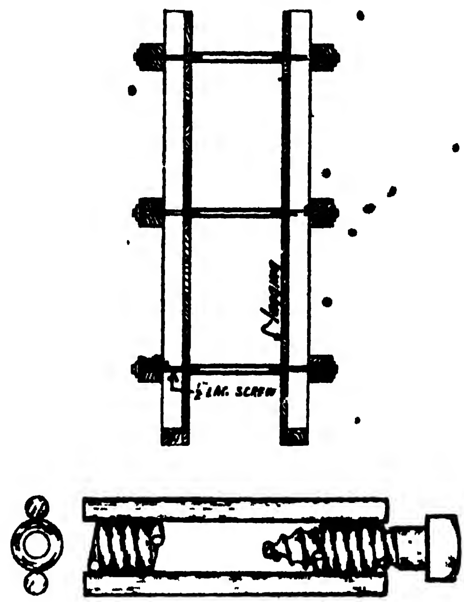


FIG. 188

Hawley Tie Bolts (*Fig.* 189).—Manufactured by the Hawley Tie Bolt Co., of Fort Worth, Texas.

These consist of three parts, a centre rod threaded each end which remains in the wall, and two end rods which are threaded at one end on the outside and at the other end on the inside; these are removed. The rods are fitted with washers and nuts.

FIG. 189

Universal Rod Couplings (*Fig.* 190).—Manufactured by the Universal Form Clamp Co., Chicago, Illinois.

These have a centre and two end rods threaded each end, the rods being connected by two hollow-threaded cone nuts. The nuts are adjusted to space the forms, acting as

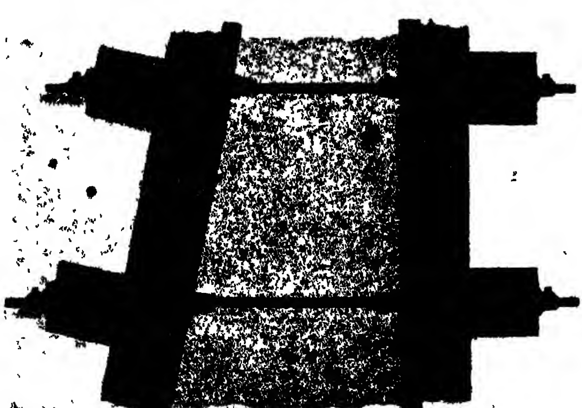


FIG. 190.

spreaders, and are removed by a wrench fitting into a square socket at the large end of the nut. The ends of the rods can be fitted with nuts and washers or "Universal" form clamps (see below). The

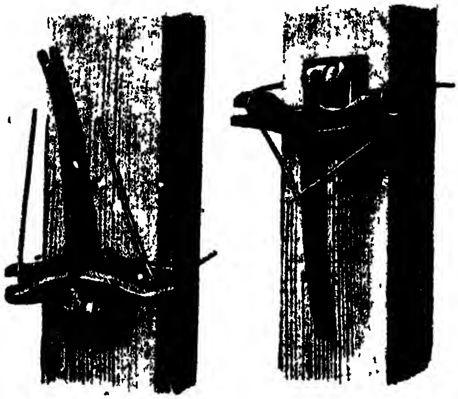


FIG. 191

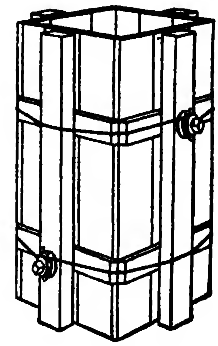


FIG. 192

advantage of the clamps is that the outer rods need only be threaded 1 in. at one end and can be any length, allowing for use on various thicknesses of timber.

Universal Wire Clamps (*Fig.* 191).—Manufactured by the Universal Form Clamp Co., Chicago, Illinois.

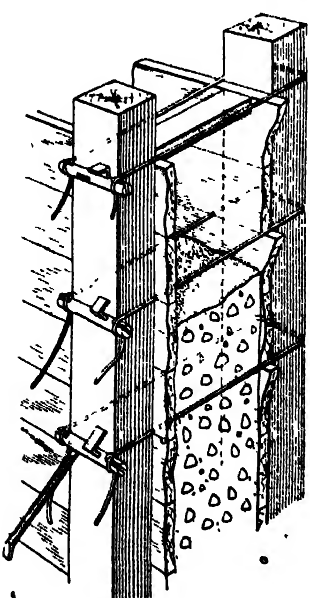


FIG. 193

These are used with ordinary wires and save labour in tying, which is done from the outside of the form. The clamp locks itself.

Bullis Form Clamp (*Fig.* 192).—Manufactured by the Washington Steel Form Co., Washington, D.C.

This is a casting used on the outside of the form for tightening the wires. The ends of the wires are passed through the clamp, which is turned until the wires are tight, and then nailed to the stud or wale. The same clamp can be used for holding column forms, as shown, and is useful for round column forms.

M. & M. Clamps (*Fig.* 193).—Manufactured by the M. & M. Wire Clamp Co., Minneapolis, Minn.

The ends of the wires are inserted in the end slots and the clamp is turned by a handle until the wire is tight; an angle

iron is then dropped into the centre slot to hold the clamp in place.

Marion Wall Tie (*Fig.* 194).—Manufactured by the Marion Malleable Iron Works, Marion, Indiana.

The malleable iron tightener is attached to a wire passed through one side of the form and a bolt is screwed into it from the face side; or two tighteners may be used as shown, with the wire entirely within the wall. The bolts govern the tension in the wire and allow for alignment, and are withdrawn after the concrete has set. They are made in five sizes, taking from ½ in. to 1 in. bolts.

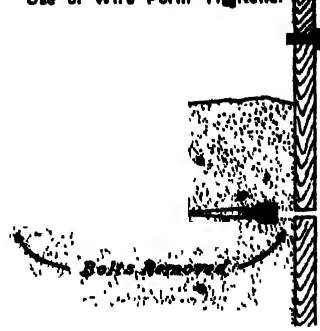


FIG. 194

Marion Couplings and Spreaders (*Fig.* 195).—Manufactured by the same firm.

The coupling is used to allow the centre rod to remain in the wall and the outer rods to be removed. The cone-shape spreaders are used between the coupling and form to space the forms.

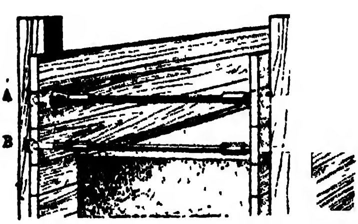


FIG. 195

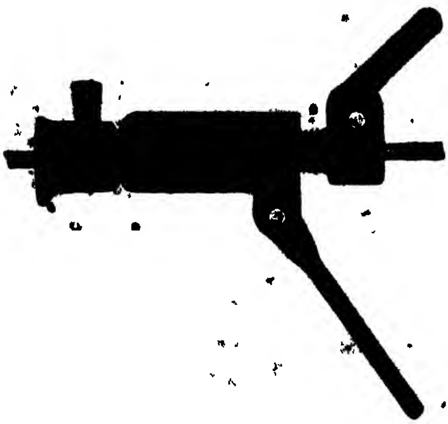
Column Clamps. When bolts are used for holding column forms, several different standard lengths of bolt must be carried in stock to fit different sizes of columns, or special long bolts with long threads must be made. Instead, plain round rods, ⅜ in. to ⅝ in. diameter, or unthreaded bolts with clamps, are commonly used. For light work, one end of the rod may be bent over once at 90 degrees and twice at 180 degrees to form a head, using a clamp on one end only. For heavier work clamps are used on both ends. To save timber in yokes, and labour in erection and stripping, there are all metal column clamps which are adjustable to different sizes of column.

Universal Column Clamp (*Fig.* 196).—Manufactured by the Universal Form Clamp Co., Chicago, Illinois.

This clamp consists of a casting with a flanged bearing surface and an oval-shape hole through the centre, with shoulders. The clamp is slipped over a plain rod or bolt and a set-screw depresses the rod between the shoulders, preventing movement of the clamp on the rod. The clamps are made in five sizes to take from ¼ in.


FIG. 196

to $\frac{7}{8}$ in. round rods; their holding power is as great as the rupture point of the rod and the bearing area is proportioned to give a safe stress on the timber. They can be used also in the same manner for tying wall forms, and with U-shape rods bent over I-beams for carrying suspended forms in structural steel fireproofing. To draw the form into line a tightening wrench, working on the principle of a hollow screw jack, is supplied (*Fig.* 197). A rod puller, made by the same firm (*Fig.* 198), is a useful device for removing the tie rods from a wall. It will exert a pull of 10,000 to 15,000 lbs., depending on the length of the pipe lever arm used, the pull being in a straight line.



197

Symon's Column Clamp (*Fig.* 199).—Manufactured by the Symons Clamp & Manufacturing Co., Chicago, Ill.

FIG. 198.

FIG. 199.

This clamp consists of two pairs of pivoted arms of mild steel, perforated with holes for adjusting to different size columns. It is held by dropping 50*d*. steel cut nails through the overlapping holes and a malleable bracket. The form is squared automatically, and the clamp is released quickly by hitting the nails with a hammer. They are made in several sizes, each with a wide range of column size.

M. & M. Column Clamp (*Fig.* 200).—Manufactured by the M. & M. Wire Clamp Co., Minneapolis, Minn.

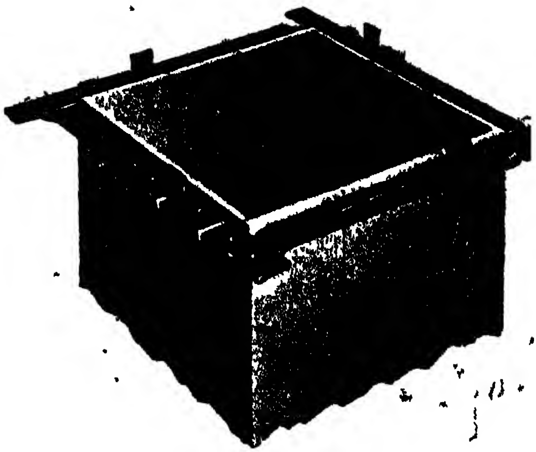


Fig. 200

This clamp has four slotted arms through which wedges are driven to adjust and hold the arms in place. The wedges allow for continuous adjustment and take-up in the timbers. They are made in two sizes for columns 10 in. to 25 in. and 14 in. to 36 in., using 2-in. sheathing.

Sterling Clamp (*Fig.* 201).—Manufactured by the Sterling Wheelbarrow Co., Milwaukee, Wisconsin.

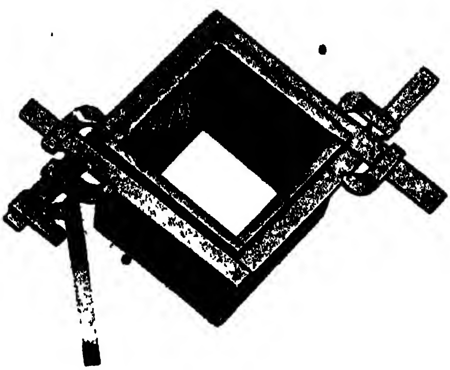
This is a band type of clamp consisting of a 14-ft. length of 16-gauge band steel, 1¼ in. wide. To one end of the band is attached a malleable iron ratchet clamping head riveted to and pivoting on a hand lever. The band is passed around the form—which may be square, round or octagonal—and the end is drawn through the clamping head. The ratchet is worked until the band cannot be drawn tighter, when the lever is pressed back against the form—putting a crimp in the band to prevent it slipping—and is locked

Fig. 201

by inserting the handle in an opening of the buckle or locking device.

O.D.G. Clamp (*Fig.* 202).—Manufactured by the O.D.G. Co., Owensboro, Kentucky.

There are two units, each consisting of two steel bars hinged together with two case-hardened steel castings on each unit. The castings grip the bars at any point and are tightened by a lever arm. They are made in four sizes for columns 24 in. to 42 in. square, varying by 6 in., and each size is adjustable down to a 6-in. square column.



Insley Clamp (*Fig.* 203).—Manufactured by Cowan Hulbert, London.

Used for column, beam and girder forms and anywhere where a clamp is required, it consists of a forged steel shank and dog. Made in three sizes to hold forms up to 18 in., 24 in., and 34 in. wide.

FIG

Universal Band Clamp (*Fig.* 204).—Manufactured by the Universal Form Clamp Co.

These are used for round and octagonal columns, and consist of a steel band 1½ in. wide fastened to the shaft of a winch containing a pressed steel frame. A slot in the loose end of the band slips over a pin in the frame and the clamp is tightened by a wrench, a ratchet holding the band at the required tension.

W.A.K. Column Clamp (*Fig.* 205).—Manufactured by the W. A. Kuhlman Co., Toledo, Ohio.

This is a band clamp with the band

FIG. 204.

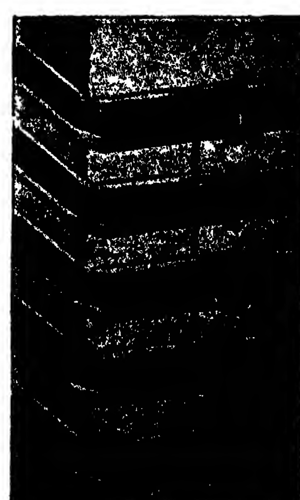


FIG. 205

steel separate from the clamp itself. Twenty-two-gauge strap-iron 1¼ in. wide is recommended. One end is folded back about 4 in., passed through the slot in the clamp and anchored with an ordinary spike. The iron is then drawn around the form and passed through the opening in the drum; the tension is drawn up with the drum nut, which is locked with a dog.

Adjustable Shores.—Shoring of forms is quite a large item of the total form cost. For each shore, including the wedges, there are at least seven individual pieces of timber required. These have to be cut and fitted, and before the shore is finally adjusted many labour operations are necessary. When they are finished with, either they have to be knocked apart and the posts used for another purpose or they are stored intact for the next job, which, however, may require shores of different height so that they have to be cut down or spliced or new shores made.

To save material and to reduce the labour involved in making and setting shores, adjustable shores are now largely used and carried as permanent plant.

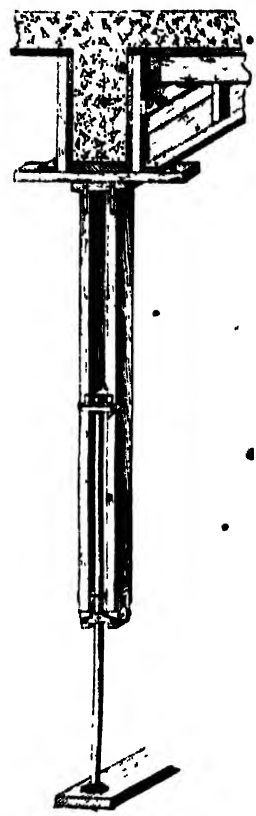


FIG. 206.

Rooshors (*Fig.* 206).—Manufactured by the H. W. Roos Co., Cincinnati, Ohio.

This shore consists of two vertical timbers with headpiece, which slides on a concrete-filled pipe between them. A lock at the bottom of the timbers automatically closes as the load is applied, gripping the pipe with heavy iron jaws. There is also an emergency lock for locking the shore either before or after erection. One man can handle a shore. It is set upright, the timber portion is raised to approximate height, and a jacking device is slipped on the pipe and final adjustment made, the shore locking automatically. A blow of a hammer releases the shore. They are adjustable from 8 ft. to 14 ft., and weigh 65 lbs. The safe working load is 3,000 lbs.

Symons Shore (*Fig.* 207).—Manufactured by the Symons Clamp & Manufacturing Co., Chicago, Ill.

This shore has an adjustment of 5 ft. with a final screw adjustment of 2½ in. It consists of a steel T-shape post fastened to a square steel base and having a collar at the top through which the timber post passes, and is carried on a casting which slides up and down the post. One leg of the post is notched and one side of the casting engages these notches and is held by a steel wedge. The other side of the casting has a screw adjustment for final adjusting, which can be done under full load.

Universal Adjustable Shore (*Fig.* 208).—Manufactured by the Universal Form Clamp Co.

Four steel angles, strapped and riveted, form a metal cage which receives the 4 in. by 4 in. timber post. The angles are notched to receive a standard steel **T** which carries the post. At the bottom of the angles is a $1\frac{1}{4}$ in. jackscrew having 3 in. of travel, which sets in a round revolving base. The **T** is inserted in the slots giving the nearest required height, and final adjustment is made with the jack by applying a wrench to the lower part of the screw, which is hexagonal. The total adjustment is 4 ft., and it is guaranteed to carry a safe load of 12,000 lbs. To use with these shores the same firm manufactures a steel Tee head which slips over the timber post for carrying beam and girder bottoms (*Fig.* 209).

FIG. 207.

FIG. 208.

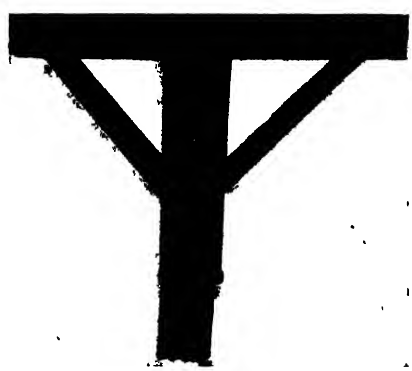


FIG. 209.

Atlas Adjustable Shore (*Fig.* 210).—Manufactured by the Roos-Meyer-Hecht Co., Cincinnati, Ohio.

This shore consists of two side members of 2 in. by 4 in. timbers with a timber head fastened to them by galvanised steel straps, sliding over a pipe post. At the bottom of the timbers is a locking plate or dog of hardened steel which engages in a canting position on the pipe, being thrown into this position by a coiled spring. The shore can be adjusted up or down by a jack slipped over the pipe.

O.D.G. Adjustable Shores (*Fig.* 211).—Manufactured by the O.D.G. Co., Owensboro, Kentucky.

The steel shore consists of two steel channels 5 ft. long, with steel foot-plate, ties and head-plates, which receive the timber post. The

post is held in place by two steel curved gripping dogs with teeth that bite into the timber. A simple locking device and release key enable the post to be lowered ¾ in., relieving the shore of the load when stripping. The post is raised and adjusted by a raising lever which slips around the post, the fulcrum resting on a ledge of the shore head. The range of adjustment is 4 ft.

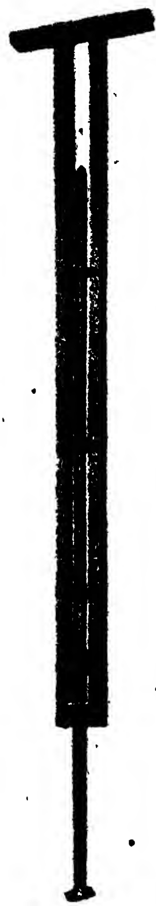


FIG. 210.

FIG. 211.

FIG. 212

The same firm also manufactures all-metal adjustable shores consisting of two telescoping pipes 1½ in. and 2 in. diameter. The gripping device consists of a malleable casting fitted with two hinged locking-dogs, fastened to the top of the larger pipe and gripping the smaller one. A raising lever raises the inner pipe to the desired height, where it is automatically held (*Fig.* 212). Various types of heads are made depending on the purpose for which the shore is to be used.

Dayton Shore (*Fig.* 213). Manufactured by the Dayton Sure Grip and Shore Co., Dayton, Ohio.

Two timber side-pieces with a timber head-piece, all strapped together, slide over a pipe, which passes through a casting on the bottom of the timbers. The pipe has holes in it through which a pin passes to hold the timber portion of the shore. The bottom of the pipe is fitted with a screw-jack for making final adjustment up to 6 in. The range of adjustment is 6 ft.

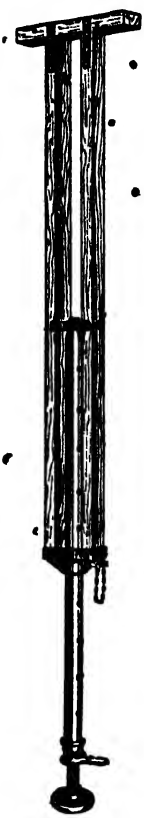


FIG. 213

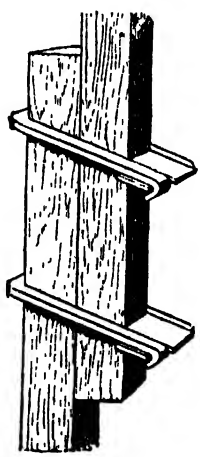


FIG 214

FIG 215.

Timber Clamps.—It is often necessary to increase the height of shores when the story height increases. This is commonly done by splicing the shore with cleats, and unless done carefully the method will give weak shores. In order to make the splice, timber has to be cut up into short lengths, with consequent waste. To eliminate this waste and the labour of splicing timber, clamps are often used. As the spliced posts overlap, any lengths on hand can be used without waste, as they are not cut. They can be used successfully also for erecting scaffolds.

Rogers Timber Clamp (*Fig.* 214).—Agents, Richmond Screw Anchor Co., Brooklyn, N.Y.

This is a U-shape steel clamp with a lockplate that bites into the timber. Two clamps per shore are used, one being removed after the concrete has begun to set. They are made in three sizes, for 4 in. by 4 in., 3 in. by 4 in., and 2 in. by 4 in. timbers.

M. & M. Splicing Clamps (*Fig.* 215).- Manufactured by the M. & M. Wire Clamp Co., Minneapolis, Minn.

This clamp consists of a steel frame which slips over the shores, with a steel wedge attached. Driving the wedge in with a hand axe locks the timbers. The standard size is for rough 4 in by 4 in.'s, the wedge driving down to $7\frac{1}{2}$ in. A special clamp is made with a filler hinged to the frame so that rough and dressed shores can be spliced together.

CHAPTER XXI.

PLANNING THE WORK.

THE common procedure of sending some timber and nails to the job and telling the foreman to get it built as quickly and as cheaply as possible is neither fair to the foreman nor conducive to efficiency and economy. If he happens to be a particularly good form builder and organiser the results may be satisfactory, but he will be an exception. More often than not there will be trouble somewhere, the forms will be too weak, cost too much, waste too much material, hold up other parts of the work, result in expensive finishing, etc., all of which could be avoided by a little careful planning ahead of the work and co-operation between the office and foreman in charge. The foreman's duty is primarily to organise labour and to get the work done. He should not in addition be expected to make form designs, draw sketches for the carpenters, and attend to details that could be done more efficiently in the office.

The busy contractor can only make periodic visits to his various jobs, so that while he is away he must be assured that the forms are being built sufficiently strong, economically, and with proper attention to details and planning of the work for maximum speed. The only way he can be sure of this is to have standard methods of building forms for different types of construction and to lay-out the work, design the forms, and make sketches of the several units in the office. The foreman then builds his forms according to blue prints as he would any other part of the structure, and he can devote most of his time to supervision of labour, in which capacity he can make the most money for his employer.

It is not meant that the foreman should have no say in the planning of the work, in the methods used, or in the details of construction. These matters should be decided upon in consultation with him in the office before the work starts; afterwards his main duty is to carry out the work according to pre-arranged plans and schedules. A few hours spent in the office will often save many lost days in the field.

The foreman is, in most cases, a better mechanic than the contractor or his engineer, and he can often suggest improvements in methods and details of construction which after trial can be incorporated in the standard plans. On the other hand, the foreman must be willing to co-operate with the office when he is asked to use different details than he has been accustomed to use. Continuous use of a certain method is apt to make a man think there is none other as good or better. Modern construction

demands speed. Speed can only be attained by planning every step in advance and working to definite schedules.

The cost of formwork in a reinforced concrete building is approximately 20 per cent. of the total cost of labour and material. The whole progress of the work, and to some extent the cost of other items, will depend on the efficient handling of the forms. It should therefore be given the attention it deserves.

There are three distinct field operations to plan: (1) assembly; (2) erection; and (3) stripping. Each is dependent on the other, on the design of the forms, and the speed schedule for the job. On a small job the general foreman can look after all operations, but on a job of any size he should have a sub-foreman attending to each operation.

Speed Schedule.—After fixing the time allowable for the completion of the concrete frame, the number of days per floor or portion of a floor are determined To maintain this rate of progress sufficient timber must be provided, depending upon the length of time the forms must be left in before stripping. Footing and foundation wall forms can always be used at least twice. One complete floor of column forms will be sufficient, as they can be stripped in two to three days. One complete floor of slab panels and beam sides is generally sufficient, though a floor-and-a-half may be necessary if the area is small.

Two floors of beam and girder bottoms should be provided, as the shores should be undisturbed for eight to ten days. As the second floor will probably be completely formed before the first floor shores can be removed, two-and-a-half floors of shores are generally necessary; this will allow work to go ahead on the third floor without waiting for shores from the first floor.

If the building covers a large area but is not very high, each floor can be divided into two parts and each treated in the same way as a separate story.

In order that the erection of the forms can start at the earliest possible moment, the timber must be assembled into panels in advance of the time they will be required. For maximum speed of erection and stripping as much as possible of the carpenter work should be done when making up the panels.

Office Detailing of Forms.—After deciding on the speed schedule required and the method of building the forms, all parts or "panels" that can be assembled in advance, and timbers that have to be cut to length, should be sketched out on paper, showing all the details necessary and the number of pieces required of each kind. This may seem like a lot of work, but if the designs and details are standardised and the detailing done according to a system it can be done very quickly.

After a method of construction has been found to be satisfactory and to give low costs there is no reason why it cannot be made a standard for all similar work. Standardisation will improve the efficiency of the workmen, as they will soon learn how to go ahead without waiting to be told what to do. It will enable closer costs to be kept and better com-

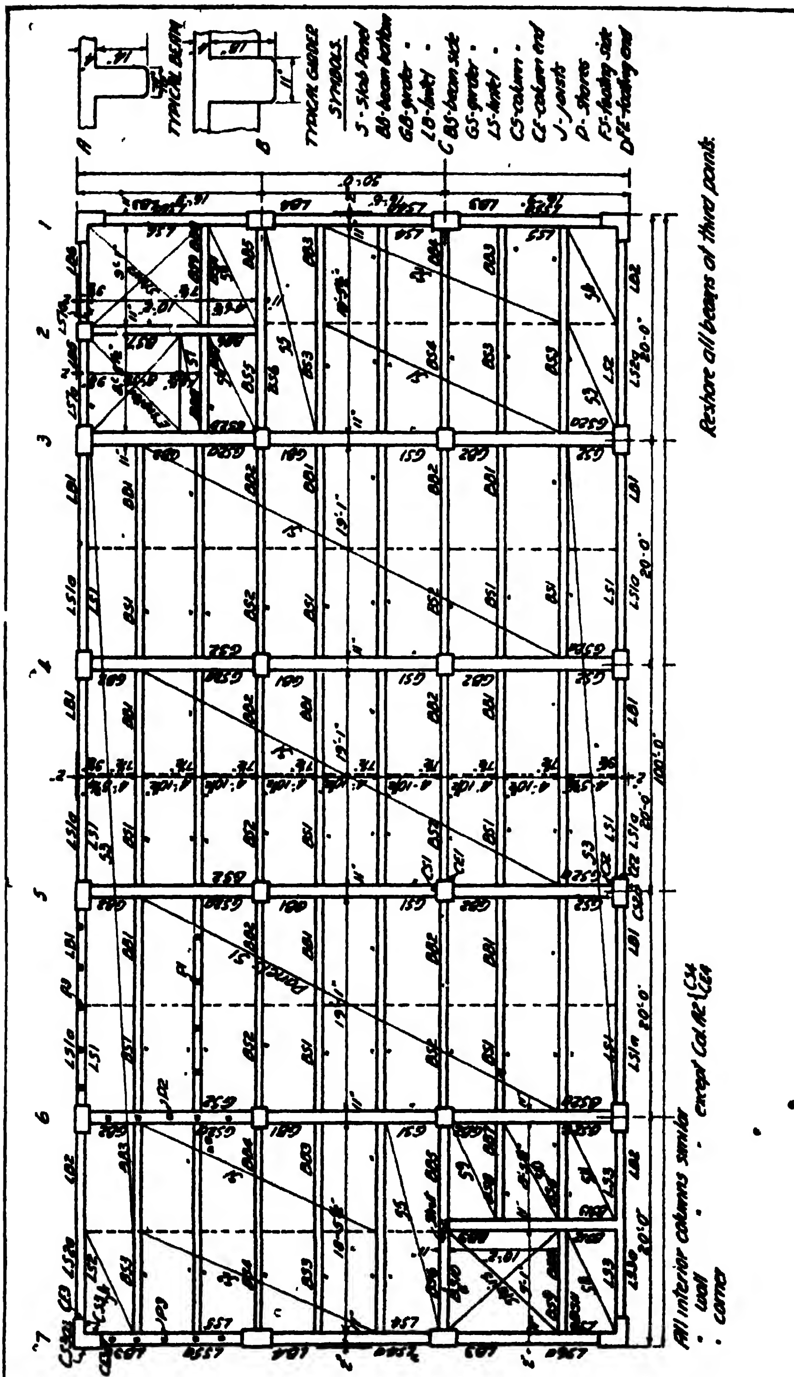


FIG. 216.—KEY PLAN TO PANELS FOR 5-STORY BUILDING.

parisons to be made of the output of the workmen and the efficiency of the foremen. It will reduce the waste in material to a minimum, since timber ordered in sizes and lengths economical for one method of construction may be quite uneconomical if another method is used. Detailing the panels in the office will eliminate mistakes in the field and much waste time of expensive workmen, since the assembly carpenters will have their work laid out for them on paper and they will not be waiting for instructions on the sizes required, the method of building, and the number of panels to make. The foreman's time will be conserved for organising. The detail sheets will show the stock lengths ordered for the different panels, so waste of timber will be prevented.

The detail sketches and number of pieces required will also be necessary for the intelligent ordering of the timber.

The detail sheets should be made in the order in which they are required on the job, unless there is time to make up the whole set before they are needed. Rough sketches are first made of the typical panels and timbers and the number of each calculated so that the bill of material can be got out as soon as possible. Some allowance must be added for waste, breakage, and repairs.

The sketches are then drawn out in more detail on special detail sheets, showing all the information necessary for the building of the panels, the number required, the stock lengths ordered, the changes necessary on other floors and any notes that will aid the erecting foreman.

Many of the panels necessary can be tabulated on one sheet, as they will be built similarly, only varying in dimensions. For instance, all beam sides will be similar; their length and depth may vary, but one sketch will do for all. Of course, here and there special panels will be required, for which individual sheets must be made out.

It is possible to standardise these detail sheets so that from the tracings white prints can be made on which the details are filled in. Sketches need not be made to scale, as all necessary dimensions are shown. Sheets should be a convenient size, not larger than 8 in. by 10 in., and preferably smaller. Only one kind of panel should appear on each sheet.

A standard system of symbols is necessary in order to identify the panels. The symbol of each panel is marked on the sheet and also on the finished panel These symbols are tied in to a key plan, which is used for erection purposes. One key plan is sufficient if all floors are alike, since any necessary changes in the forms can be shown on the detail sheets. If the floors are different a key plan will be necessary for each different floor.

This plan is drawn, taking a section between floors looking up at the floor above, and shows the outlines of the concrete members only. All panels required are indicated on this plan by symbol. Although originally drawn to a large scale it should be reduced to a convenient size for handling on the job.

A simple system of symbols easy to remember is S for slab panel, BB for beam bottom, BS for beam side, CS for column side, FS and FE

for footing side and end, etc. A number added to the symbol indicates the location on the key plan, and if it is necessary to show the floor on which a panel is to be used the number of the floor precedes the symbol; for instance, 3BB12 indicates the beam bottom for beam 12 on the third floor. Panels that are similar are given the same symbol.

The method of making the detail sheets can best be shown by taking an actual example, say, of a reinforced concrete factory. It is assumed that the reader has followed the previous chapters, showing how to obtain the sizes and spacing of timbers, the provisions to make for easy stripping, clearances and other details. *Fig.* 216 shows a key plan for a typical floor of a 5-story factory, 50 ft. by 100 ft. in size. The lay-out and beam sizes on all floors and roof are kept the same—which is good practice—so that one key plan does for all floors. The column sizes change from floor to floor, and the changes required to the panels are shown on the detail sheets. Following the key plan are shown typical detail sheets, covering most of the panels required for the building. Twenty sheets would cover all the panels necessary. More notes are given than would be required if the carpenters and foreman were familiar with the system. The stock lengths for ⅞-in. boards are not given, because this size is generally ordered in random lengths varying from 10 ft. to 16 ft. Since much of this has to be cut up in short lengths there is little waste.

The system of construction used is (*see* Chapter IX) slab panels resting on loose joists carried on continuous ledgers; beam bottoms carried on girder or column sheathing; beam sides and girder sides built with about 1½ in. clearance at each end for bevelled keys. Additional sheets can indicate the size and number of braces, ribbons, etc.

Assembly.—The timber pile, saw table, and assembly bench should be placed together in a location where there is plenty of storage space and where a minimum amount of handling of the panels will be required. The relation of each to the other should be such that the timber will pass in one direction through the sawmill and assembly bench to the stock-pile, which should be nearest the structure.

If the job is large enough a sub-foreman should be in charge of all assembly, from delivery of the timber to the sawmill to delivery of the finished panels to the erecting carpenters.

Copies of the detail sheets go to the saw operator and to the assembly carpenters. The foreman picks out the timber required, which is carried to the saw table, where it is sawn to the required lengths, and passed on to the assembly bench where it is made up into panels.

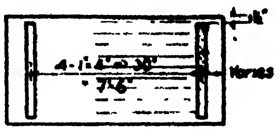
Battens, cleats, etc., should be sawn first, so that they are ready at the bench when the boards are received.

The foreman must study the order in which the panels will be required so that there will always be a sufficient number of each kind required ready for the erectors.

As the panels are completed they should be marked with their symbols, oiled, stacked, and moved to their location in the building as required.

SHEET No 1

JOB No. MARK S1 to S11

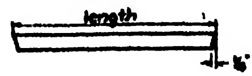


Boards 7/8"·5½", Battens 1"·4" at 30"
Bevel 2 sides, 1 end
Cut from 10 stock, split waste 7½ boards for battens
Sufficient for 1 Floor provided

MK	SIZE	No	REMARKS
S1	4'-10"·9'-6"	30	All panels same on
2	4'-10"·8'-10½"	8	all floors
3	4'-5"·9'-6"	14	
4	4'-5"·8'-10½"	2	
5	4'-8"·9'-2½"	4	
6	4'-6"·8'-5"	1	Bevel 2 S 2 E
7	2'-0"·8'-5"	1	" 1 S 2 E
8	4'-6"·9'-0"	2	" 2 S 2 E
9	4'-8"·8'-5"	1	" "
10	4'-10"·8'-5"	1	" "
11	4'-5"·8'-5"	1	" "

SHEET No 2

JOB No MARK J1 to J11

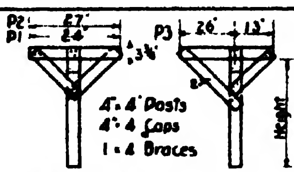


All Joists 2"·6". ¼" clearance allowed
Bevel ¼" each end
Space all joists at 30"
Cut from 10' stock
Sufficient for 1 Floor provided
J1 goes with S1, J2 with S2, etc.

MK	SIZE	No	REMARKS
J1	4'-8½"	180	same all floors
2	"	24	
3	4'-3¾"	30	
4	"	6	
5	4'-6¼"	14	
6	4'-4½"	4	
7	8'-3½"	1	Carry on B5 & B7
8	4'-4½"	8	
9	4'-6¼"	4	
10	4'-8½"	4	
11	4'-3¾"	4	

SHEET No 3

JOB No MARK 1P1-2-3
2P1-2-3

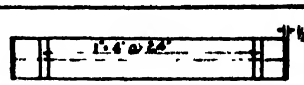


Cut from 12' stock, use waste for braces
Use 2"·12" mudsills 1st Fl
Sufficient provided for 2½ Floors

MK.	SIZE	No	REMARKS
1P1	9'-5"	165	Use under beams at 4'0"
2P1	8'-10"	250	on c 4 to each span
1P2	9'-1"	60	Use under girders of
2P2	8'-6"	90	2'-3, 5 to each span
1P3	9'-3"	80	Use under lintels of
2P3	8'-8"	120	ends 2'-9", 5 to span
			sides 3'-6".

SHEET No 4

JOB No MARK BB1 to BB9



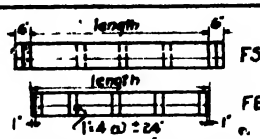
2" Plank for 7½" use single plank, for 11" use 2 - 5½" plank with 1"·4 cleats at 24"
Bevel ends ½"
1" Clearance allowed
Sufficient provided for 2 Fls
Cut from 18' and 20' stock,

MK	SIZE	No	REMARKS
BB1	7½"·19'-0½"	36	
2	7½"·17'-11"	12	Add 2'-2" 1st Fl, 4'-3"
			8'-4", 12'-R4
3	7½"·18'-5"	16	
4	7½"·17'-7"	4	Add 1'-2", 2'-3"
			8'-4", 10'-R4
5	11"·17'-7"	4	ditto
6	11"·14'-11"	2	Add 4'-4"
7	7½"·8'-3"	6	
8	7½"·9'-0"	4	
9	11"·15'-3½"	2	Bevel 1 End

FIG. 217.

SHEET No. 5

JOB No MARK FS1-2-3-1a-2a
FE1-2-3-1a-2a



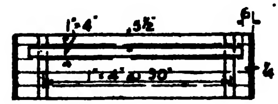
7/8" × 5½ or 7½" boards 1" × 4" battens at 24"

Sufficient panels for ½ Footings

MK	SIZE	NO	REMARKS
FS1	14" × 8'-1⅝"	8	Interior Step Foot'g
1a	14" × 5'-4¾"	8	
FE1	14" × 8' 0"	8	
1a	14" × 5'-0"	8	
FS2	11" × 6' 9⅝"	12	Wall Col Step Footg
2a	11" × 4' 8¾"	12	
FE2	11" × 5'-6"	12	
2a	11" × 3'-5"	12	
FS3	14" × 4' 7¼"	4	Corner Cols and
FE3	14" × 4'-6"	4	Col A2

SHEET No. 6

JOB No MARK BS1-4-6-8-9
11-2-3

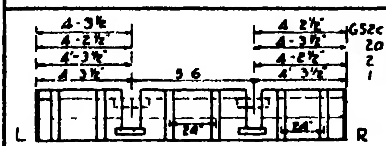


Boards 7/8" × 7½", Battens 1" × 4" at 30"
Ledger 1" × 4", Keys 2" × 4" × Depth of side
Tack 2 keys to each side
Bevel each end ¼"

MK	SIZE	NO	REMARKS
BS1	14⅞" × 18'-9"	36	Same all Floors
2	14⅞" × 17' 7"	12	Increase length 2" on
			2nd, 4'-3rd, 8'-4th 12 Rf
3	14⅞" × 18'-1½"	16	Same all Floors
4	14⅞" × 17'-3	4	Incr 1'-2nd, 2'-3rd, 8'-4th, 10 R
6	16⅞" × 17'-3"	2	ditto
8	12⅞ × 8'-1½	6	Same all Floors
9	17⅞" × 8-9	2	ditto
11	12⅞" × 8'-9"	2	ditto

SHEET No 7

JOB No MARK GS1-2-2a-2c

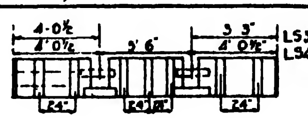


Boards 7/8" × 7½", Battens 1¼" × 4" at 24" Cleats 2" × 2"
Provide 2 keys 2" × 4" × 18⅞
Bevel Ends and Sides of Openings ¼"
Openings 8" × 14⅞" except GS2c - 8" × 12⅞"
Tack cleats over openings

MK	SIZE	NO	REMARKS
GS1	18⅞" × 14'-1"	8	Add 1" L & R-2nd, 2" L & R-3rd
			4" L & R-4th, 6" L & R-Rf
2	18⅞" × 14'-0"	6	Add 1" L-2nd, 2" L-3rd
			4" L, 4" R-4th 6" L-Rf
2a	18⅞" × 14'-0"	8	Reverse to GS2
2c	18⅞" × 14'-0"	1	Same as GS2

SHEET No 8

JOB No MARK LS4-5

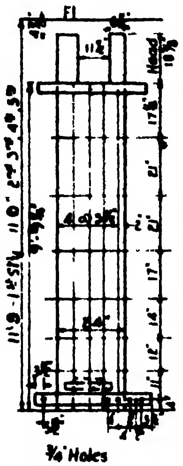


7/8" Boards 2-5½", 1-7½", Battens 1¼" × 4"
Cleats 2" × 2" × 16", Provide 2 keys 2" × 4" × 16⅞"
Bevel Ends and opening sides ¼"
Tack cleats over openings
Openings 8" × 14⅞"

MK	SIZE	NO	REMARKS
LS4	16⅞" × 13'-7"	2	Same all Floors
5	16⅞" × 12'-9½"	2	Clearance of interior
			Col only

FIG. 218

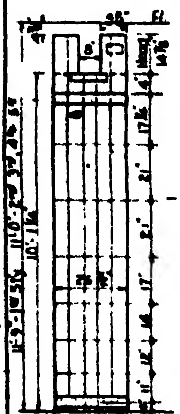
SHEET No. 9
JOB No.
MARK CS1



16 req'd
Sheathing 1⅛" × 5½"
Yokes 4" × 4" × 3'-6"
Head separate, tack to form, bevel edges ⅛"
Reduce to 22" in 2nd S'ty
20" " 3rd "
16" " 4th "
12" " 5th "
Reduce 9" at bottom, 2nd
Bore all holes req'd
Provide cleanout door, and attach with cleat

Cut sheathing from 10' stock, yokes from 14' stock

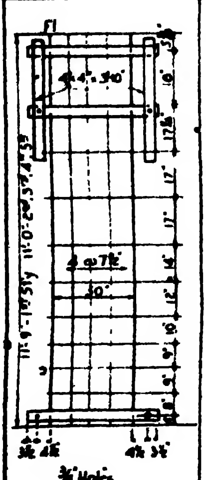
SHEET No. 10
JOB No.
MARK CE1.



16 req'd
Sheathing 1⅛" × 5½"
Yokes 3" × 4" × 2'6½" on side
Head separate, tack to form, bevel edges ⅛"
Reduce to 24½" in 2nd S'ty
22½" 3rd
18½" 4th
14½" 5th
Reduce 9" at bottom, 2nd

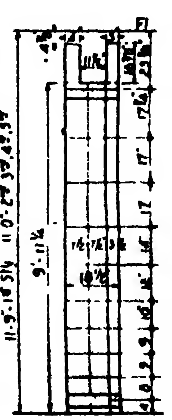
Cut sheathing from 12' stock, yokes from 14'

SHEET No. 11
JOB No.
MARK CS2a



12 req'd
Sheathing 1⅛" × 7½"
Yokes 4" × 4" × 3' 10"
Attach 2 extra 4 × 4" × 3' 10"
Remove bottom yoke and reduce height 6" on 2nd S'ty
Cut sheathing from 12' stock, yokes from 16'

SHEET No. 12
JOB No.
MARK CE2



24 req'd
Sheathing 1⅛" × 7½"
Yokes 2" × 4" × 18½" on side
Head separate, tack to form, bevel edges ⅛"
Place 3" dimension outside
Reduce 4" on 1 side in 4th S'ty
Reduce 9" at bottom in 2nd S'ty
Cut sheathing from 10 stock, yokes from waste

FIG. 239.

When they will not interfere with other work they can be carried directly from the bench to the building, piling in small stacks conveniently located for the erectors. Braces, wedges, ribbons, mudsills, etc., which may have to be cut but not made up are also handled by this foreman.

Erection.—All panels, timbers, etc., necessary should be delivered to the erecting carpenters as required, so that they will not have to leave their work to fetch material or wait for it.

The key plan is the foreman's guide to erection, but at the same time he must be familiar with the building plans. He should also have a set of detail sheets to show any changes necessary to the forms on upper floors.

Methods and order of erection have been mentioned in Chapter IX. The work must be planned so that each operation is kept in advance of the succeeding one. The placing of slab panels should closely follow the setting of beam sides, so that the steel workers can follow on close behind the forms.

A bench should be set up on each floor where necessary changes to the forms can be made. It is seldom economical to send panels back to the sawmill to make these changes.

The erecting foreman must look after the supply of nails, wire, bolts, clamps, etc.

Stripping.—The order of stripping the panels, as mentioned previously, depends on the system used. Cleats and keys that have to be removed before stripping should be nailed on the panels again to prevent loss.

As panels are stripped they should be cleaned and carried to a convenient place for hoisting, and distributed to their right location on the floor above or to the bench if they have to be changed or repaired.

This foreman will have charge of re-shoring, the method of which should be indicated on the key plans. On his ability to strip carefully, without damage to the forms, will depend the cost of repairing before they can be used again. Panels that are not to be used again should be cleaned, taken apart, nails removed, and the timber carried to stockpiles. Careful stripping and final dismantling of the panels can save the contractor much valuable timber.

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